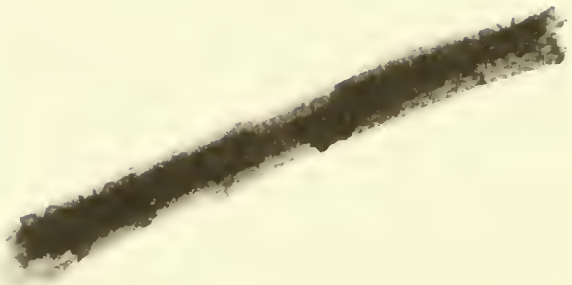

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## ERRATA.

Page 228.—In heading of Paper 371, for "November, 1888," read "November, 1887."

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVII.—July, 1887.)

# ADDRESS AT THE ANNUAL CONVENTION AT THE HOTEL KAATERSKILL, NEW YORK, JULY 2, 1887,

BY WILLIAM E. WORTHEN, President Am. Soc. C.E.

I was much gratified by my election as President of the American Society of Civil Engineers, a numerous body of men who fully represent the profession, and with whom I have been for a long time connected. In assuming the office I undertook the duties of the position, which, however honorable, are not entirely without responsibility. I now proceed to fulfill one of the requirements by delivering an annual address, which is supposed to be a *resumé* of what was done last year and the progress of the work of engineers, with some impressions of my own experience and my opinion with regard to the future.

I have collected sundry statistics from newspapers, magazines, etc., and although I have not credited them to the different parties, I yet follow the rule of the society in their publications, in not assuming the responsibility for the facts and opinions advanced, nor shall I back my opinions against facts if they are hereafter found in opposition thereto.

There is little of the catastrophic in engineering science. It is essentially an evolved science; the works and facts from which it is

evolved have been the accumulation of centuries; shrewd guesses or judgments worked out; a copying of examples; changing construction according to the teaching of experience, and application of varied material dependent upon location and requirements. Without going back into remote history, when the term engineer was not known, although there were wonderful examples of engineering construction, let us consider the growth of the science up to its present standpoint. Who built the first bridge? The change from wooden to iron structures is recent, yet how full is the record of steps. Who conceived the first railway? Its growth from the strap railway is wholly within our recollection. How many have contributed to make the turbine the wonderfully efficient and economic water-motor it now is? Who first thought of the application of steam to the purposes of power? How many worked at the construction and improvement of boilers and steam engines? We can pick out parts that were an essential improvement, and give the names of the men who first designed them, but the great body of the machine is made up of parts which have been constructed by mechanics without name, and not only the parts, but the machines to make the parts uniform and interchangeable.

Many an invention conceived before its time could be unearthed from patent records; either because it was not then needed, or the tools which made it economically and practicable were not to be had. How much development of industry has been given by the lathe, planer, and shaping machines? Seldom, if ever, in the lapse of a single year can we record any striking novelty which can be set down as a step in progress. It takes years to complete and practically test. Like the statue of "Liberty Enlightening the World," which is one of the accomplished facts of last year, it took many years to design, construct, and raise it. It is unfortunate, however, that in illuminating the world, no provisions were made for illuminating the statue itself.

The name Engineer is of quite recent date, and is very mixed in its definition; the Architect preceded it. Architecture was always considered among the ancients as one of the liberal arts, but the name of architect was not given till a few centuries ago to artists charged with the construction of buildings. The man of the profession before the architect was the master of the works, a practical builder; and work comprehended the entire building and its furniture. It is probable that the religious houses, which contained all that there were of men of

letters, savants, and scholars on the Continent, furnished not only the men who designed and directed the monastic constructions, but also the civil ones, and perhaps military. The master of works became the head of a corps of workmen which made a part of the free corporations which regulated wages and hours of labor, and defined rights and duties. Once outside of the cloister, work went on with great rapidity. The master workman was the acknowledged head; his name was connected with his work, and he was buried within the most prominent church he constructed. But the necessities of variation in designs and details soon brought in the artist and the architect, and something beside the construction of buildings was necessary.

In 1560–1641, Sully, Grand Master of Artillery and Superintendent of Finances of Henry IV of France, was the first that appreciated the service that could be rendered by a body of geometricians and mathematicians, and he organized a *Corps de Génie*, which took the title of "*Ingénieurs Ordinaires du Roi.*" This organization remained unchanged until the time of Colbert and Vauban, the latter the "*vrai père de Génie Civil et Militaire.*" L'Ecole Mezieres, established in 1748, was the first institution devoted to the development of the sciences. This was the period when the expression the "*Corps de Génie*" was introduced into the army. In England, the distinction between engineers became that of civil and military. It was evident in the discussion of the paper of Mr. William Metcalf, M. Am. Soc. C. E., on Steel, before this society, that in the application of this material to the construction of guns and to civil purposes, no distinctions were drawn between the civil and the military, and the graduates from West Point are able civil engineers and fulfill that duty under the direction of the War Department. In general, the term engineer is supposed to come from engine, and the term engineer embraces not only the man who designs and constructs it, but the man who runs it. I suggest that there should be some distinction; that engineers should be such as are eligible to our society, and that the men who run and take charge of engines or machines should be called *enginers.* I want to reclaim, perhaps, a little more of "*le génie*" for the profession than for the mechanical artisan; for skill, experience, and industry may make a very good workman, but to this must be added more general and extended information in the application of materials to necessities, mathematics to calculate, and drawings to express; and, more than this,

a sentiment of purpose, a feeling for the work. A true engineer is as much born as a poet. Could we divide the application of the term into engineers and enginers it would be distinctive, and I would drop the term civil as embracing too large a profession, and adopt prefixes as military, hydraulic, railway, mechanical, steam, electric, and other divisions, when these are adopted as specialties. With regard to military engineering, I know of no particular progress of recent date. The War Department has undertaken to clear out Gedney's Channel, in New York Harbor, by pump dredging, and it is satisfactory to report that the work last year retained its depth without filling in, and a new contract has been entered into at about half the price of last year and is successfully progressing.

Hydraulic Engineering.—Constructions for the Use of Water for Power, City and Town Supplies, Hydraulic Mining and Irrigation.

In the census of 1880 will be found tabulated accounts of the water-power of the United States, developed and undeveloped. Attention is called to the low percentage of increase of water power in comparison with that of steam:

| | | | | |
|---|---|---|---|---|
| In 1870, Water power, | 1,130,431 H.P. | Steam, | 1,315,711 H.P. |
| In 1880, " | 1,225,379 " | " | 2,185,418 " |
| Percentage of increase, | 8.40 " | " | 79.77 |

It will be seen that the percentage of increase of steam is very much in excess of that of water, but the latter still is not an inconsiderable amount, 95,000 horse power, nearly equal to the power of the City of Lowell, per year. A convenience of access to business and labor centers has tendered very largely to the increase of steam power. And although there are immense water powers yet undeveloped, and the cost of steam power is largely in excess of that of water power, yet position and its relations have decided in favor of steam. And where large manufactories have been commenced on water power. and where the water power has become exhausted, or very irregular, the business has been extended by the use of steam, rather than by a diversion of it to a new place for water. In 1844 I assisted Mr. U. A. Boyden in conducting his experiments on the first Fourneyron turbines at the Appleton Mills, Lowell, Mass. This wheel gave 76 per cent. useful effect from the water, whilst some breast-wheels at Lowell, which Mr. James B.

Francis, Past President Am. Soc. C. E., and myself tested some time before, gave from 60 to 66. Mr. Boyden afterwards constructed many other turbines throughout the country, introducing many improvements in design and construction. One at Lawrence gave over 90 per cent. of effect. Boyden came of a very remarkable family, and had, above all men that I ever knew, that desirable genius, the genius of industry. He filled in all his time with scientific work and research. His relaxations consisted almost entirely of changes of subject. His improvements in water-wheels increased the water power at Lowell about one-fourth over what was obtained by the former wheels, and gave him the means of pursuing other investigations of light, heat, sound, etc. His mode of experimenting was that now followed by Mr. Francis at Lowell and Mr. Mills at Lawrence. With the impulse given by Mr. Boyden, water-wheel shops were established in different parts of the country. The wheels were tested and improvements made, until now water-wheel construction has passed out of the hands of the engineers into those of manufacturers, who furnish wheels ready made, simple, efficient and strong, guaranteed to give 80 per cent. effective power.

### Water Supply of Towns.

There is a very large increase in the number of town and cities supplied and additions to work already constructed. The great improvement in the matter of aqueducts has been in the application of wrought-iron for cast-iron mains, and also to take the place of masonry and tunneling. Large wrought-iron pipes for conducting water to turbines have been in use from the time of the introduction of the Boyden turbine wheel, but it has been left for California to largely extend its use for the conducting of water for hydraulic mining, irrigation and town supply, with a boldness of design and execution which has astonished the engineer. The method of hydraulic mining was introduced or invented in California in 1852. The supply of water for these jets at first was conducted through hose made of heavy duck cotton cloth, which was strengthened by outer nettings of cordage when the pressure was large. In 1853 an ingenious miner laid in his mine a line of pipe consisting of ordinary stove-pipe, made of very thin sheet-iron, lightly fastened together with cold rivets, with the joints united stove fashion. This pipe was five or six inches in diameter. As hydraulic mining increased in magnitude the sizes of these supply pipes also increased, the

diameter for main lines for a large mine being from 22 to 30 inches. These pipes, as a rule, are made at the mine. The iron is made from .055 to .134 inches (No. 16 to 10 Birmingham gauge) in thickness, with a double row of cold rivets for the longitudinal seam when the pressure is to be large. As a protection against rust, each joint is immersed for several minutes in a bath of boiling asphaltum and tar. They are now used for permanent conduits both for conducting water to mining districts, across deep mountain gorges, and also for the supply of cities. San Francisco, a place of some 300,000 inhabitants, receives its water through two lines of such pipes, and a third pipe, many miles in length and of large diameter, is now being laid for an additional supply. Within the last year one of the large shops in San Francisco has increased and improved its plant, so that they are now able to turn out weekly a mile of pipe 44 inches in diameter. Of the quantity of water necessary per inhabitant there is considerable difference of opinion, but there is no doubt in the general opinion, that in the present use of water there is an extensive waste.

Of the quality of water for a town supply there is a great difference of opinion among chemists and scientists. M. Gerardin (*Rapport sur l'Alteration, la Corruption et l'Assainissement des Rivieres*) says "the distinction between healthy waters and polluted ones cannot be detected by color, smell, taste, or chemical analysis. Water can be very much polluted without giving any smell. Such are the waters of paper-mills, starch and sugar-works at the leaving of the works. Many natural waters which are prescribed in the case of sickness are very offensive in smell. Chemical analysis does not sufficiently explain the alteration of waters. If a perfectly healthy water be inclosed some time in a bottle, well-corked, its elementary analysis always manifests the same results, and, notwithstanding this, it has undergone radical change and has lost all its good qualities."

Chaptal says "that those who make a chemical examination of waters only analyze its dead body." "Water is healthy when animals and vegetables of a superior organism can live in it; on the contrary, a water is affected when animals die, and it can only nourish infusoria. There is no better means of establishing the character of water than by determining if in this water fish and aquatic plants can live. If fish die there and plants perish there, the character is certainly polluted, and the water should not be used for domestic purposes. On the contrary,

if fish and aquatic plants can live in it the water can be considered good." There is an old French proverb, "*Il n'est pire eau que l'eau qui dort*" (There is no worse water than that which sleeps) which conveys the general impression with regard to the sources of supply. A running stream supplies the best water, and when reservoirs are necessary, the one from which the water is drawn directly should be as small as possible, and the water from superior reservoirs should flow down into it, so that the water may be well aerated in its course. Organisms are not dangerous because microscopic, and they are not, as a rule, dangerous. "It is only," as an eminent scientist observed, "when germs of disease are introduced from the outside, that the waters become dangerous," and organisms actually contribute largely to the purification of the water. If, by the doctrine that water once polluted is always polluted, it is to be understood that these pollutions are accumulations from century to century, then, for the preservation of our species, our own organisms must be changed to adapt us to the circumstances, and in our new evolution we should not be suited to the air and water of the Garden of Eden and its suburbs. M. Gerardin, in the work above quoted, speaking of *algæ*, says they are endowed with a very active respiration. "The oxygen which they accumulate decomposes rapidly organic matter, and they contribute largely to the purification of waters polluted by organic matters in decomposition. From the biological examination of the water of various streams that are polluted by paper-mills, and sugar, starch, and glue-works, as it leaves the works, and at different stages of the stream below, he finds that *cryptograms* and *algæ* first begin to appear, and thus up through the variety of forms to the higher and higher organisms, until fish appear in their waters and plants along their shore." Experiments in aquaria confirm this conclusion. I would refer to the report on pisiculture of James B. Ferguson, Commissioner of the United States at the Paris Universal Exposition of 1878, on the value of the *algæ*. He says: "Oxygen in its native state, resulting from the respiration of vegetable, and especially from microscopic *algæ*, is very soluble in water. Atmospheric air introduced mechanically into water is not dissolved there, or if so in very small quantities." He quotes also from the Superintendent of the Aquarium his experiments on the aeration of waters supplied, as follows:

"The water of the Vanne, taken almost from its source, contains consequently little air and no organic germs; hence, although very

useful for the cleansing and purification of the basins, it was not suitable to favor the rapid development of a cryptogamous vegetation upon the sides of the tanks. I substituted, therefore, the waters of the Seine, the color of which was, at that time, green on account of the abundance of microscopic germs, sporules of *algæ*, *conferva*, and, in fact, all those vegetable aquatics—for the most part microscopic—the presence of which is requisite for the continuous oxygenation of water intended to receive fish. In truth, upon water, as well as upon air, has been imposed that great law of nature which counterbalances and equalizes the animal and the vegetable kingdoms, making the one the indispensable auxiliary of the other.

"Fish, as well as animals on land, absorb oxygen by respiration, and give out carbonic acid, the quantity of which, always increasing in the place which they inhabit, would not fail to produce asphyxia if the aquatic vegetable were not present to produce a chemically inverse effect. Under the influence of solar radiation the green immersed particles absorb carbonic acid, decompose it, assimilate the carbon and exhale the oxygen, which appears upon their surface in innumerable small bubbles, supplying the fish with the respiratory element, renewed and endowed with a vital action so much the more efficacious because just brought to life, and possessing a solubility much greater than that of oxygen drawn from the atmosphere.

"Perhaps there is even a formation of bioxide of hydrogen, which would explain this coefficient of solubility as being so much greater than that admitted by chemists. However that may be, for me the fact is now incontestable that to establish in an aquarium a hygienic medium suitable for the life of aquatic animals, an abundant vegetation must there be developed, which should frequently undergo the action of solar radiation."

Most successful aquaria are constructed with what may be termed closed circuits, not drawing water from an exterior supply, except to make up waste. Water is used over again and again by aerating it in its circulation. Mr. C. B. Brush, M. Am. Soc. C. E., has written a paper which will be read before the Society, of experiments in forcing air into water by means of air-pump pressure at the Hackensack works, as he claims, with success.

Whether the cucumber taste of the Croton water be due to the *spongilla lacustrina* or to *nostochonia*, it seems to me to be rather due to the

want of circulation, and the decease and decay of some organism by which the balance of aquarial life has not been properly kept up. The pabulum has failed, and perhaps it might be remedial to introduce a little sewage, as it has been shown by analysis that the sewage at Troy improves the water of the Hudson, and that the water supply of Albany is better than that above. The *algæ* called frogs' spittle, *Batrachosperma moniliforma* (I might here remark, the smaller the organism the bigger the name), will always be found in the purest spring water. I am pleased to see them, and feel more sure of the purity of water by the presence of such common organisms than by their absence, understanding that within the microcosm there is a class of organism which depends on the usual food supplied by nature. Disturbance means disease, if not death. *Post hoc* is not always *propter hoc;* don't at once ascribe disease to the presence of an organism; and when you see crows or buzzards hovering around the carcass of a horse, don't come to the conclusion that they have killed a valuable animal and shoot the crow or buzzard.

In the early times of gold-mining in California, hydraulic mining was the rule; but with the increase of value for agricultural purposes, the disturbance of rivers, and the covering of land with *debris*, this has been found very objectionable, and water has taken a value for irrigation. But with irrigation developments the necessity of defining water-rights has become indispensable, and a law was passed creating the office of State Engineer, and duties were assigned opening a large field of inquiry over an immense territory. The field of work is in the nature of a physical survey of the State, combined with certain industrial, social, and other inquiries of a broad scope, and necessary to be prosecuted over a wide field.

The first part of the report has now come out, and it is for sale by the State of California. "It is introductory to the study of problems of irrigation, and is composed of a series of studies of irrigation development in the three countries of France, Italy, and Spain, from whose experience we propose to learn something for our own immediate purposes." We have referred to this report not only for its value in matters treated, but because like necessity exists in all our States to define the rights of water-users, whether for power or town supply; the drainage of land for agricultural purposes and sometimes irrigation; and in the discharge of polluted matter from streams. In whom does the title to

water lie? Is it the first user? When the large city takes the entire flow of contiguous rivers and streams, whence is the fast increasing suburban population to be supplied, and once taken has any town or city the right to deprive others of this by waste? In 1883 an act was introduced into the New Jersey Legislature to provide for the appointment of commissioners to determine on plans for the storage of any of the waters of that State for the purpose of furnishing to cities and towns a joint water supply. With regard to this, commissioners were appointed and a small amount of money was provided for expenses. The report was made, but no further action was taken. The commission is still in existence, but without power or money. In 1886 an act was passed by the Commonwealth of Massachusetts, requiring from the Board of Health a report on the condition and means for protecting inland waters. A report has been made under this act. This year a new bill was introduced to establish a Board of Water Commissioners, to provide for the establishment of water companies and the general supervision of drainage connected with inland waters. Such boards are necessary in every State, not only to define the uses of water and the removal of waste within the limits of the State, but also there are interstate rights, and you can readily understand that the majority of such boards should be hydraulic engineers and with such powers as the State can delegate.

## Steam Engineering.

In 1857, Mr. Charles W. Copeland and myself made the first experiments on pumping engines at Belleville, N. J., and Hartford, Conn. (Published in the *Brooklyn Water Works*, and issued by the City of Brooklyn). It may be remarked that the evaporation of the boilers, 10 to 11 pounds of water per pound of coal from water at 100 degrees, will still be considered a very fair evaporation. But the duty per 100 pounds of coal was 55 000 000 to 62 000 000 on the Hartford engine and 68 000 000 to 73 000 000 on the Cornish engine at Belleville, now very much below the duty of first-class pumping engines. From the above experiments, Mr. James P. Kirkwood, Past President Am. Soc. C. E., felt authorized to require by contract for the first Brooklyn engine a duty of 60 000 000 pounds feet per 100 pounds of coal. It is not now unusual to require 100 000 000 pounds feet, or one-horse power for a little less than two pounds of coal per hour, and this taken on

the water end of the machine. First-class pumping engines are now invariably compounded, and compounds have also been largely introduced in stationary engines and with largely economic results; of their exact comparative economy we need more tests.

When steam was first introduced for the heating of cotton-mills, a boiler was placed at each end of the mill, with automatic water-feed by means of balance stone float. Later the boilers were concentrated within one house and the steam distributed through ranges of immense mills. The original steam-heating pipes were cast-iron, about 3 to 4 inches in diameter, and suspended from the ceiling; later, with the introduction of wrought-iron pipe, the pipe was carried around the walls of the mill near the floor, but at present the wrought-iron steam-pipes are suspended from the ceiling. In mills where large quantities of steam are used for other purposes than power, it is common to pass the steam through a steam engine for power, and then use the exhaust for drying, boiling, and heating. In 1866 I made a report to the Merrimack Manufacturing Company, of Lowell, on the economy of using steam first for power at the cotton-mills and afterwards the exhaust for their print works for the purposes of heating, boiling, etc., and reported on the advisability of passing the steam through engines of 1 200 horse power, and using the exhaust steam afterwards in the print works, with a back pressure of about six pounds, and that the amount of coal properly chargeable to power in this case would not exceed one pound of coal per horse power per hour. It was a success, and of its present extent the superintendent of the works writes me as follows: "We have only one pair (34 by 72) condensing engines on our premises; all the others (something over 100 cylinders) exhaust into a system of mains, from which we draw all the heat we require. What we do not require for heat we take into the condensing cylinders before mentioned, and reconvert into power." Some time since there was a boom in the steam-heating of towns and cities from central stations, and many plants were constructed many of which have gone out. There are still two in New York City, one of moderate size, comparing fairly well in extent with that of a large cotton-mill plant; the other is very large and is still increasing its distribution. This result seems to argue the survival of the fittest.

### Railroads.

There has been substantial progress in the extension of railroads, in

the construction of new roads, and doubling and quadrupling the old ones. The estimation of construction for the year is 10 000 miles. The combined locomotive has been tested in Europe, but not sufficiently to afford a fair test of its practical value, and has not been put in operation here. The public has been greatly exercised by numerous accidents in the destruction of cars and passengers by fire, and our legislators have called for a safer means of heating and lighting than at present in use; in New York one year is given for the change. The two great questions of automatic freight-train brakes and automatic freight-car couplers have made great progress during the past year. The freight-brake tests which were made at Burlington, Iowa, in July of last year, have been supplemented by a second series this year, the two series together being by far the most complete tests of the kind which have ever been made. The results demonstrated that only air brakes are adapted to the requirements. The prominent part of this year's tests was that taken by electricity as an actuating agent for applying the brakes, with results showing great benefit from its use, both in quickness of stop and in avoiding the dangerous shocks which result from quick stops with long trains. The electric application being absolutely simultaneous throughout the train, no shock whatever results. The occasional failures of the electric apparatus during the test indicated that further improvements are still needed, but as electricity simply supplements the air, which acts as usual in case of an electric failure, the probability seems strong that such electric application will be a feature of the train-brake of the future, while a percentage of efficiency was shown in this year's test, much greater than was ever shown before. As a safeguard for human life, rapid progress has been made toward the introduction of automatic freight-car couplers. The Burlington brake tests of this year and last show that it was on the whole decidedly beneficial to eliminate loose slack between trains which has tended to exclude couplings of the link and pin type, and the final step has now been taken by the formal adoption by the Master Car Builders' Association of the Janney hook coupler and others of the same type coupling with it as standards of the association, by a vote of 33 to 14.

## Street Railways.

The entire service of street railways was for many years by the means

of horses, and we quote as to the cost of working the following returns taken from the sixteenth annual report of the Massachusetts Board of Railroad Commissioners: Highland, Lynn and Boston, Metropolitan, Middlesex, and South Boston; number of horses owned, 6 158; number of miles run, 11 287 196; number of passengers carried, 68 196 776; average number of passengers carried per round trip, 43.4.

From the above it appears that the stable average daily distance traveled by the above horses equals 10.04 miles.

The average number of passengers per round trip being 43.4, per single trip equals 21.7. Averaging them at 140 pounds, equals 3 038 pounds. Add weight of car, 4 800 pounds, equals 7 838 pounds.

With the great extension of street roads there have been found great inconveniences in this horse service, not only in the cost, but in the location of immense stables within populous neighborhoods. The usual traction force has been reckoned to be about $\frac{7}{10}$ per cent. in movement, and from 6 to 7 per cent. in starting on level grades. This will afford us a means of comparison between different forms of motors.

Although the cable was used on the London and Blackwall Railroad for many years and for heavy trains, it was superseded by the locomotive, but it has of late years been revived in this country for the distribution of city travel and with great success.

Chicago has 20¼ miles of cable road in operation and 22 under construction; Cincinnati, 8 miles; Kansas City, 4 miles in operation and 30 under construction; New York City, 10½ in operation; Philadelphia, 18 miles; St. Louis, 6 miles; San Francisco, 33½ miles now in use; Melbourne, Victoria, has several miles; London, 3 miles; and four years ago a road was built in New Zealand. The first cable road in San Francisco was opened in 1873. This was the famous "Clay Street Hill Cable Road," where the grades were so steep that property was almost valueless. The cables had no trouble with the grades, 17 feet in 100, where horse power was useless, and the road was an immediate success.

Mr. W. H. Paine, M. Am. Soc. C. E., has furnished me with the following facts in regard to the cable on the Brooklyn Bridge, which he laid: "The practicability of applying cables to the hauling of large heavy passenger cars in trains at considerable speed with shorter headway, greater regularity and less danger from accident than any other method known, is here demonstrated.

"The durability of the cable, and of the grips and dies has been here so great as to remove a most serious obstacle to cable traction.

Speed of cable, 10 miles per hour.
Length of cable, 11 000 feet.
Size of cable, 1½ inches in diameter.
Weight, 3½ pounds per lineal foot.

"The first cable was taken out last November, after having been in use three years and 41 days, having hauled 1 675 790 cars, 48 108 567 passengers, or 22 196 436 tons. The heaviest cars weigh 16 tons when empty, the heaviest load of passengers about ten tons. The lightest cars weigh 11 tons when empty. Three cars are now hauled in one train. Shortest headway about 1¼ minutes. More than 100 000 passengers have been carried in one day. The grips will last for years with few repairs. The dies or lining of the grips last more than four months, sometimes exceeding five months. The speed of the cable, which is 10 miles per hour, could be increased, but without increasing the carrying capacity of the road."

### Electric Railways

for street passenger traffic seem now to be a fixed fact. In Europe electric street railways are now carrying at the rate of 3 000 000 annually, and probably a little in excess of that number here, and they are extending. The first built was in Berlin, Germany, 1881, 1½ miles in length. In this country they date from 1885. The following are the places in which they are established: Baltimore, Md.; Los Angeles, Cal.; Port Huron, Mich.; Windsor, Canada; Detroit, Mich.; Appleton, Wis.; Scranton, Pa.; Denver, Colo.; Montgomery, Ala. In all but one of these the dynamo is driven by a steam engine, but in Appleton, Wis., there is a road of 4¼ miles where water power is used. You may recollect that an eminent divine once remarked as an evidence of design in the works of the Creator, that great rivers were placed near great cities. It were fortunate for the purpose of lighting and heating if great water-powers were placed in the vicinity of large, populous districts. It has not yet been sufficiently decided how far power can be carried by means of electricity, nor the size of the conductor and the loss in conducting it. With the short roads now in operation it has been said that the percentage of effect on the car is 50 to 55 per cent. that of the steam engine

providing the power. In this respect it is superior to the cable, which is fast superseding horses.

Few of the present engineers recollect the experiment of the old atmospheric railway of Samuda, which ran from Dublin to Kingston, about 1½ miles, successfully for many years; and was later tried in a larger way from London to Croydon and from Paris to St. Germain. It failed and was superseded by the locomotive. It may be that for a moderate distance the atmospheric road may become practicable, but for the present our hopes of a successful motor, especially for city and town roads, are based on electricity. So much attention at least has been directed to this subject, not only with reference to power, but to light and heating, that a new branch of engineering has sprung up and has inaugurated itself into a society like our own as the American Institute of Electrical Engineers. From our knowledge of them and their claims I feel satisfied that they are to be a worthy branch of engineers. In our own country capitalization in electrical districts, telegraphic, telephonic, electric lighting, and electric power, amounts to a sum of at least $350 000 000. There are in America about 700 000 miles of telegraphic wire; last year 32 000 000 conversations were held on the telephone; there are ever 150 000 arc lamps, and incandescents verging on a million; and at least 15 000 electric motors for fans and sewing machines.

### Bridges.

The first Howe truss was built across the Connecticut River, at Springfield, Mass., in 1840. It was a wooden bridge with double intersections. Smaller bridges had been built in the same line of road, but this was a very important bridge at the time. Howe was a carpenter and the bridge was designed by average judgment, and the construction of one bridge made the precedent for another. Engineering science was not then applied to the calculating of stresses. Undue confidence in the form of truss, and the successful construction of many bridges in wood led to the Ashtabula accident. The strong desire for the construction of iron bridges at minimum cost, and the high price of iron, led to the construction of weak designs of this material, and postponed its adoption for many years. With the reduction of the price of iron, and a better understanding of the principles of construction, iron bridges have almost entirely superseded the wooden ones.

The forms have been calculated and tested by years of practical use,

and they are furnished by engineering bridge companies in lengths to suit, and guaranteed to resist the stresses required. The finding of the jury in the Bussey Bridge disaster on the Boston and Providence Railroad, is acknowledged to have been just, but this is not likely to occur with bridges as at present furnished, and subjected to careful inspection on the part of railroads. The limit of span has not yet been reached, nor perhaps the admissible height of skeleton piers, nor any form of truss or suspension established as the best. Cantilevers, from the ease of their erection without false-works, are largely used, but æsthetically in form they do not compare with the arch; yet extended use and knowledge of their "fitness for function" will soon reconcile one to the form.

### Gas Engineering.

Much has been done by improvements in mechanics and chemistry to increase the production of gas and reduce its cost. Water-gas is largely manufactured, and its uses extended in forging and welding. It is to be hoped that it may be distributed by mains throughout cities for the purposes of heating buildings and cooking. And although from experiment it has been demonstrated that water-gas has but 65 per cent. of the thermic value of the coal from which it is manufactured, yet when the cartage and the storage of coal is taken into account, together with the curtailing of the hours of consumption by gas to the time necessary for the purpose, and the economy in the application of gas, there seems to be a fair presumption that, sooner or later, water-gas will be used for heating and cooking. In fact, from the great waste of heat in lighting by gas, and its cost, there perhaps might be some economy in converting the heat of water-gas into power through an engine, and into light by a dynamo, rather than directly by carburetted hydrogen into light.

### Sanitary Engineering.

Of late years this branch of the profession has been recognized. It embraces the construction of sewers, the removal of sewage and its purification, or rather offense from it, and the plumbing and heating and ventilation of houses. The construction of sewers has long belonged to the department of hydraulic engineering, sometimes by the combined and sometimes by the separate system, the general aim being to remove sewage speedily from the town or city, and to points remote from settled

neighborhoods, and, if possible, discharge it into a large body of flowing water. Of the offense in the matter of smell from sewage there is no doubt; but that it is so dangerous to life and detrimental to health as is often claimed, I have just doubt from a large experience and from evidence of experts before court.

I have said in the earlier part of my address that the engineer is born, like the poet, sculptor, painter or any other artist, but that he is cradled on palpable material and nursed on facts, not metaphysical ideas, of "I think, therefore, I am," or even the syllogism of the second order of difference, "I think I think, therefore I think I am;" but "I am an engineer, and must think and work." It is not necessary for him that he should believe either in the corpuscular or the wave theory of light, in the atom whose weight is 1, the first digit in the 22d place of decimals of a gram, that pressure is due to the immense velocity of atoms in motion, or that even bodies fall by attraction of gravitation or by some later discovered force. If these present theories enable him to recollect facts, they serve their purpose. It is sufficient for the engineer to know the laws of force to be able to resist or use them; $v = gt$ or $v^2 = 2gs$ supplies him with the laws of gravitation even if he cannot understand the why.

The works of the engineer involve higher responsibilities than any other profession; he is to build, suitably, with materials at his disposal, solidly and securely for the prevention of accidents and for the protection of life, and economically in cost. He is to draw from his own and the experience of others, but he should not try experiments at another's cost without a full understanding with the parties employing him.

As an expert he must testify to facts, and the party who retains him must accept his facts, as they may be brought out in evidence, whether they are for or against them. It were better that the expert were attached to the court and not retained by either plaintiff or defendant. The construction of the law depends on the ruling of the judge. The value of the facts, from his point of view, should be settled by the engineer. As an umpire between the capitalist and contractor his decision should be impartial, and if, by his own error, either party has been put to an unnecessary expense, acknowledge the fault, and, as far as possible, remedy it, but where such responsibility is taken, the price should be commensurate.

From the position of the engineer as umpire between the capitalist

and contractor, and his large employment and familiar intercourse with mechanics and laborers, he would naturally be the proper party to adjudicate in disputes between labor and employer, and perhaps it would be so were the present troubles between labor and capital. But the strife is to see whether labor cannot support another class: chapters, presidents, officers, walking delegates and the like. These troubles come up in every country and in every age; they are like the measles, whooping cough, fever, cholera, in the microcosm. Ours, at present, is an imported disease; the diagnosis of the suppression of the individual, of boycott, etc., are entirely un-American. It is not our own labor, but the imported labor that is striving, not for a chance for labor, but rather for a life without labor. There is no country in the world where labor is so well paid, well clothed, well fed, and well housed. We have taken in an immense immigration, and still are, and have endeavored, by making them citizens, to assimilate them rapidly into our body corporate. This should be a question of time. It is a common phrase to say one has bitten off more than he can chew. How much more sad our case when we have swallowed more than we can digest! The great panacea seems to be land; some of us have worked land, and think if land were given to us instead of bread it would be the gift of a stone, but if properly applied it might relieve our present labor trouble. Let every immigrant be sent at once into the country and furnished a cabin and ten acres of land, without tax, as there is to be hereafter no title to land, and support himself and family ten years on the land. At the end of that time the majority would prefer the work and wages of a day laborer, especially in the city, and all would be better citizens. Syndicates of labor are as proper organizations as syndicates of capital or of employers, and syndicates can only be met by syndicates. Properly organized, they are not expensive and are efficient; but individuals should have perfect freedom to do as they please with their labor or capital within the limits of law, and must be protected by law and a healthy public sentiment.

The Secretary has furnished the following statement of the present (June 22) membership of the Society:

| | Resident. | Non-Resident. | Total. | |
|---|---|---|---|---|
| Honorary Members | 3 | 5 | 8 | |
| Corresponding Members | ... | 3 | 3 | |
| Members | 161 | 632 | 793 | ... |
| Associates | 11 | 34 | 45 | ... |
| Juniors | 28 | 92 | 120 | 958 |
| | 203 | 766 | | |
| Total | | | | 969 |
| Fellows, 64, of whom 6 members are included above, leaving | | | | 58 |
| Subscribers to the Building Fund, 140, of whom 90 are entered in one or other of the above classes, and 11 deceased, leaving | | | | 39 |
| Total now connected with the Society | | | | 1 066 |

Whence are we to draw our future membership to maintain the ranks of our Society? In answer, a distinguished professor and engineer has kindly furnished me the following statistics:

"There are in the United States 360 colleges and universities, which, besides classical and other courses, teach technology, and have given diplomas on technical professional subjects to 1 058 graduates. There are besides 49 schools, working under the 'Land Grant,' with 593 teachers and 4 644 students, of whom about 600 were technical graduates. In addition to these there are 39 private technical institutions with nearly 300 teachers and over 7 000 students, 800 of whom received technical diplomas. Statistics of this kind are very difficult to obtain, but it may be said that our higher schools of science turn out about 2 500 men per year, and of these about seven per cent. are engineers."

It becomes us, who are conversant with the needs of our profession, to advise that before entering our technological institutes, students should have a good school training, such as is required in entering classical universities, and that within the institutes they shall know not only what to learn, but also how to learn. Then in their outer world progress they may know how to accumulate knowledge useful to themselves in their profession, and reciprocate for the information and pleasure of those with whom they are thrown in contact.

In the earlier part of my address I claimed for the origin of our name, Engineer, *le genie*—genius; but genius, inborn, is derived from the Greek *gune*, woman, and I trust that there is no profession which has a higher respect or greater love for that name than our profession, whether as mother, sister, wife, or companion in the halcyon days of youth, or, in maturer years, our origin, protection, comfort, and inspiration. We trust that they will always attend our annual outings, intended not only for interchange of views in matters of our profession, but also as a relief from business, a union for the establishment of a more general acquaintance, and cementing the bonds between the members of the American Society of Civil Engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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# ON THE CALCULATION OF THE STRESSES IN BRIDGES FOR THE ACTUAL CONCENTRATED LOADS.

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READ MAY 4TH, 1887.

The actual loads to which railroad bridges, and, in a lesser degree, street bridges, are exposed, consist of a series of concentrated loads. Therefore the most accurate method of calculating the stresses in such structures would be to take the assumed or prescribed system of actual loads, find its position on the structure necessary to cause the maximum stress in any particular member, and determine in this way the maximum stress in each piece. Up to within about ten years this method was rarely followed, the usual method (except for the floor system) having been to assume a uniform load over the entire length covered by the engines and train; or, better, such a uniform load with the addition of concentrated loads over the drivers or at the head of the train, constituting the so-called locomotive excess. And it seems to the writer that this method is as accurate as any, while much more simple than that which depends upon the actual loads; for, in the first place, the weights of loaded cars have of late years so increased as to at present almost equal the heaviest engine loads, while the weights and dimensions of engines themselves are continually changing, so that, by assuming the load to consist of the concentrated loads of two particular engines followed

by a uniform train load, we are in reality, so far as the actual service of the bridge in question is concerned, assuming a system of loading almost as fictitious as a uniform load with locomotive excess would be. Nevertheless, many of the latest and best specifications require the use of the actual loads for engines and tenders, followed by a uniform train load, and this method seems destined to be for some time a favorite one. It therefore becomes necessary to investigate the stresses due to systems of actual loads, and the object of this paper is to give a discussion of the simplest methods of finding such stresses, with the corresponding positions of the system of loads. For the sake of completeness some methods will be given which are well known to bridge engineers, while others will be explained which it is believed will be found new to the profession in this country, and some which are original with the writer. It seems to be the belief of many that it is very much more difficult to calculate with the actual loads than with the locomotive excess; a belief which the writer hopes he may show to be erroneous in a great many cases. Considerable misconception, moreover, appears to still exist with regard to the exact position of the loads causing maximum stresses in different members, notwithstanding the fact that it requires in most cases but a very few moments to determine exactly the proper position required, even in the case of some structures which may in general be called complicated, like arches and cantilevers.

The writer is not acquainted with any systematic and complete treatment of this subject in English, and he therefore offers this paper with the hope of in some measure supplying this deficiency.

Commencing then with simple cases, let us proceed to consider moments and shears.

## I. Maximum Moment at Any Point on a Beam.

### A. *Without Floor-Beams.*

Considering the beam $A\ B$ in Fig. 1, Plate I, suppose we wish to find the maximum moment at any point $c$, situated at a distance $x$ from the left abutment. Let the loads on the bridge be as represented, $G_1$ being the resultant of all the loads to the right of $c$, and $G_2$ that of all the loads to the left, $G_1$ acting at a distance $n_1$ from the right abutment, and $G_2$ at a distance $n_2$ from the left abutment. Then the moment at $c$ is

$$M = G_1 \frac{n_1\ x}{l} + G_2 \frac{n_2\ (l - x)}{l} \qquad (1)$$

If now the entire system of loads be moved a distance $\triangle$ to the left, the moment at $C$ becomes

$$M' = G_1 \frac{x(n_1 + \triangle)}{l} + G_2 \frac{(n_2 - \triangle)(l - x)}{l} \tag{2}$$

and $M'$ is greater than $M$ if

$$G_1 x > G_2 (l - x)$$

or if

$$\frac{G_1}{l - x} > \frac{G_2}{x} \tag{3}$$

But $\frac{G_1}{l - x}$ is the average load per running foot to the right of $c$, and $\frac{G_2}{x}$ is the average load per running foot to the left of $c$. Hence the rule:

If the average load per running foot to the right of the section is greater than the average load per running foot to the left of the section, then the moment at the section will be increased by moving the system of loads to the left, and *vice versa*. In other words, the maximum moment at any given point will occur when the average loads to the right and left are as nearly equal as possible. But with concentrated loads we shall be able only in exceptional cases to make the average loads on either side absolutely equal. We shall, however, always be able to place the loads so upon the bridge that with some particular load an infinitely small distance to the right of the section in question the average load to the right shall be greater than that to the left (thus showing that the loads should be moved to the left), while for the same load an infinitely small distance to the left of the section the average load to the left shall be the larger (thus showing that the loads should be moved to the right). From this it will follow that the load in question should be at the section; and from which it also follows that for the maximum moment at any section, some load must be at that section.

Suppose that with some load $P_n$ just to the right of the section, we find the average load to the right larger than that to the left; we then move $P_n$ just to the left of the section, and if now the average load to the right is still greater than that to the left, we then move the next load $P_{n+1}$ up until it is just to the left of the section, and repeat the trial, regardless whether, during the operation, new loads come upon the bridge to the right, or leave it at the left. For, when $P_n$ is just to the left of the section we move to the left, because the average load

to the right is greater than that to the left; and if, in moving the loads to the left, additional loads come upon the bridge at the right, then the average load to the right is so much larger still, and, *a fortiori* we must continue to move to the left. The same form of argument applies when loads leave the bridge to the left.

This method will not give, of course, with absolute certainty the position of loading causing maximum moments; for it sometimes happens that two or more different loadings will be found which will satisfy the condition just explained, and to find which of these gives the absolute maximum moment, each must be worked out numerically. By remembering, however, that to produce the maximum moment at any point, the heaviest loads must be as near that point as possible, the cases in which trial is necessary will be greatly reduced in number.

### B. *With Floor-Beams.*

In the case of truss bridges it is only necessary to find the moments at the joints, but at some of these there may be floor-beams, and at others there may be none. It is, however, rarely necessary to consider the floor-beams at all, for the reason that, as is well known, the moment at the floor-beams is precisely the same whether floor-beams be supposed to exist or not, while at points between the floor-beams the actual moment (when floor-beams exist) is slightly less than the moment which would exist were there no floor-beams. It is therefore in almost all cases sufficient to calculate as though there were no floor-beams at all, but as if the loads were applied directly to the girders. Methods will be soon explained, however, by which the effect of floor-beams at intermediate points may be considered if desired.

## II. Maximum Shear at any Point on a Beam.

### A. *No Floor-Beams.*

Suppose that we desire to find the maximum positive shear at any point $c$ of a beam (Fig. 2, Plate I). It is well known that for the maximum positive shear the portion $c\,B$ of the beam must be loaded, and that as a train comes on the bridge from the right, the shear at $c$ will increase at any rate until the first wheel reaches $c$. If the first wheel passes $c$ the shear suddenly diminishes by the amount of the load $P_1$, but if the train move still farther to the left it will increase again until $P_2$ reaches $c$. The question now is: Will the shear be greater when $P_2$ is at $c$ than

when $P_1$ is at $c$? This question is easily answered: Let $R$ be the resultant or sum of all the loads on the bridge when $P_1$ is at $c$, then the shear at $c$ is the left-hand reaction $R_1$ or $R\frac{n}{l}$. If now the train be moved a distance $a_1$ to the left, so that $P_2$ is just to the right of $c$, the shear will be $R\frac{n+a_1}{l} - P_1$ plus some additional small quantity $d$, which is the left-hand reaction due to any additional loads which may have come on the bridge during this advance of the train. The shear will therefore be increased by moving up $P_2$, provided

$$R\frac{a_1}{l} + d > P_1 \tag{4}$$

or (since we can almost always neglect $d$), provided

$$R\frac{a_1}{l} > P_1 \tag{4A}$$

It is only necessary to consider $d$ when these two terms are very nearly equal, which does not very often occur.

Similarly, the shear would be increased still further by moving $P_3$ up to $c$, provided

$$R_1\frac{a_2}{l} > P_2 \tag{5}$$

$R_1$ being the sum of the loads on the span when $P_2$ is at $c$.

B. *With Floor-Beams.*

Let the distance between floor-beams (Fig. 3) be called $p$. Then since the shear at any section $c$ is equal to $R_1$ minus all the loads to the left of $c$, and since no loads act on the main girder except through the floor-beams, it follows that the shear between any two adjacent floor-beams is constant. The question therefore is to find the maximum positive shear in any panel, such as $C\,D$.

As a train approaches from the right, the shear will be simply the reaction $R_1$ as long as all the loads are to the right of $D$, and it will continually increase until the first load lies just at $D$. If now the train is moved farther, the shear will be equal to $R_1$ minus the load which comes on the floor-beam $C_1$, and will either increase or decrease uniformly until $P_2$ arrives at $D$, or until $P_1$ passes $C$. The question is therefore as before: Will the shear be greater with $P_1$ or $P_2$ at $D$? This is easily determined, for when $P_1$ is at $D$ the shear is $R\frac{n}{l}$; while with $P_2$ at $D$ it is

$$R\frac{n+a_1}{l}+d-P_1 \text{ (if } a_1>p)$$

or,

$$R\frac{n+a_1}{l}+d-P_1\frac{a_1}{p} \text{ (if } a_1<p)$$

Hence the shear is greater when $P_2$ is at $D$, provided

$$R\frac{a_1}{l}+d>P_1 \qquad \text{(if } a_1>p) \tag{6}$$

$$R\frac{a_1}{l}+d>P_1\frac{a_1}{p} \text{ (if } a_1<p) \tag{7}$$

or, neglecting $d$ as before, provided

$$R\frac{a_1}{l}>P_1 \qquad (a_1>p) \tag{6A}$$

$$R\frac{a_1}{l}>P_1\frac{a_1}{p} \ (a_1<p) \tag{7A}$$

In these equations, which are extremely easy of application, $R$ represents, as before, the sum of all the loads on the bridge when $P_1$ is at $D$.

Having found, perhaps, that the shear is greater with $P_2$ at $D$ than with $P_1$ at $D$, the question may arise: Will it be greater still with $P_3$ at $D$? Calling $R'$ the sum of the loads on the bridge when $P_2$ is at $D$, the condition in this case will be easily seen to be the following: The shear will be greater with $P_3$ at $D$ than with $P_2$ at $D$, provided

$$R'\frac{a_2}{l}+d>(P_1+P_2)\frac{a_2}{p} \qquad \text{(if } (a_1+a_2)<p) \tag{8}$$

$$R'\frac{a_2}{l}+d>P_1\frac{p-a_1}{p}+P_2\frac{a_2}{p} \qquad \text{(if } a_1<p,\ a_2<p, \text{ and } a_1+a_2>p) \tag{9}$$

$$R'\frac{a_2}{l}+d>P_1\frac{p-a_1}{p}+P_2 \qquad \text{(if } a_1<p,\ a_2 \tag{10}$$

$$R'\frac{a_2}{l}+d>P_2\frac{a_2}{p} \qquad \text{(if } a_1>p,\ a_2<p) \tag{11}$$

$$R'\frac{a_2}{l}+d>P_2 \qquad \text{(if } a_1>p,\ a_2>p) \tag{12}$$

In most cases, as before, we may neglect $d$.

## III. Lines of Influence.

We proceed to explain a method by which shears and moments, as well as stresses, deflections, etc., may be calculated, and which, by its perfect generality, may be applied to the solution of a great variety of problems which could scarcely be solved in any other way.

To show what a line of influence is, let us consider first the case of moments on a beam, and we shall see how the results already reached follow very simply from the properties of the influence line.

Let $A\ B$ (Fig. 4, Plate I) represent a beam supported at $A$ and $B$, and suppose we wish to find the maximum movement at the point $c$. To do this, imagine a single load equal to unity to pass over the bridge from right to left, and at each position of the load let us lay off as an ordinate the moment which would exist at $c$. Thus, $m\ n$ represents the moment at $c$ which would be caused by a load of unity at $m$, and $c\ d$ that which would be caused by the load unity at $c$. The $A\ d\ B$, which is the influence line for the moment at $c$, evidently gives the moment for any and every position of the load; $c\ d$ is evidently equal to $\frac{x\,(l-x)}{l}$, and $A\ d$ and $d\ B$ are straight lines, which may be drawn by first laying off $c\ d$ to some scale.

Now suppose the actual loads on the bridge are as represented in Fig. 5, and that we desire to find the moment at $c$, clearly this moment will be

$$M = P_1\,y_1 + P\ \ y_2 + P_3\,y_3 + P_4\,y_4 + P_5\,y_5 + P_6\,y_6 + P_7\,y_7 \quad (13)$$

From the influence line we can therefore obtain this moment at $c$ for any position of actual loads on the span.

Suppose, again, that the load consists of a uniform load of $p$ pounds per running foot. The influence line may be used as before; for in Fig. 6, taking an infinitely small distance $d\,x$ at $g$, the load on this distance will be $p\ d\,x$, and the moment it will cause at $c$ will be $p\ y\ d\,x$, since a load of unity at $g$ would produce a moment $y$ at $c$.

Here the moment caused by the uniform load over any portion $m\ m'$ of the span will be $\int_{x_1}^{x_2} p\ y\ d\,x = p \int_{x_1}^{x_2} y\ d\,x = p\ A$, $A$ representing the area $m\ n\ n'\ m'$. The moment caused by the load over the entire span will be $p\ A$, $A$ being the entire area $A\ d\ B$, or it will be $M = \frac{1}{2}\,x\,(l-x)$, since the ordinate $c\ d$ is, as already explained, $\frac{x\,(l-x)}{l}$

The influence line enables us therefore to find the moment due to any loading whatsoever. It also shows us how much of the beam must be loaded in order to produce the maximum moment, where a single load must be to produce the maximum moment, and it may be used in a variety of problems, as will soon appear.

Suppose now we wish to find how a system of single loads must be placed on a beam to cause the maximum moment at $c$. We have already investigated this problem, but we shall now show how the same results follow from the properties of the influence line, and how we are thus enabled to draw some further conclusions. In Fig. 7, Plate I, let $A\ d\ B$ be the influence line as before, and let $G_1$ and $G_2$ be the resultant loads to the right and left of $c$ respectively. Now if the entire system of loads be moved a distance $d\,x$ to the left, then any load $P$ lying at $m$, and which at first caused a moment represented by $P\ y$, will now cause a greater moment, namely $P\ (y + d\,y)$, or its moment will be increased by $P_1\ d\ y = P\ d\,x$ tan. $\alpha_1$. The same will be true of all the loads $G_1$, while all the loads $G_2$ will have the moments decreased by $P\ d\ x$ tan. $\alpha_2$. The resultant moment at $c$ will therefore be increased by moving the system to the left, provided

$$G_1\ d\,x \text{ tan. } \alpha_1 > G_2\ d\,x \text{ tan. } \alpha_1, \text{ that is if}$$

$$G_1 \text{ tan. } \alpha_1 > G_2 \text{ tan. } \alpha_2, \text{ or if}$$

$$\frac{G_1}{l - x} > \frac{G_2}{x}$$

as we showed on a previous page.

Influence lines may be drawn not only for the moment at a point, but for the shear, or for the stress in any bar of a frame; or for the deflection, or any other function of the loads which we desire to investigate. From the theorem just proved, one very fertile principle follows, which finds numerous applications, namely, this:

*Whenever the influence line for any function is composed of two straight lines, like those in Fig. 7, Plate I, then the maximum value of the function in question, for a system of concentrated loads, will occur when the load covers the length A B when some load is at c, and when the average load on A c is as nearly as possible equal to that of B c.*

*In other words, the loading causing the maximum value of the function will be the same loading which would cause the maximum moment at c on a beam of length A B.*

Before proceeding to the applications of this principle, let us consider some other lines of influence.

*Moment; with Floor-Beams.*—As before remarked, we most frequently have to find the moment at the points where the floor-beams are supported, and at such points the moment being precisely the same,

whether there are floor-beams or not, the influence lines are the same. At points between the floor-beams, such as $c$ in Fig. 8, Plate I, the influence line, as will easily be seen, is $A\ H\ G\ B$, the moment $c\ d^1$ being somewhat less than the moment $c\ d$, which would exist were there no floor-beams. The simplest way of drawing this influence line is to lay off $c\ d$, $A\ d$, and $d\ B$ as though there were no floor-beams, and then join $G$ and $H$.

*Lines of Influence for Shear.*—Suppose we wish to study the shear at a point $c$ on the beam $A\ B$ in Fig. 9. The influence line will evidently be $B\ d\ e\ A$, $c\ d$ being equal to $\frac{l-x}{l}$, and $c\ e = -\frac{x}{l}$. The straight lines $B\ d$ and $A\ e$ are parallel, and $d\ e = 1$. This line tells us at once that with a uniform load the maximum positive shear occurs when only $c\ B$ is loaded, and the maximum negative shear when only $A\ c$ is loaded; and, further, that

$$\text{Maximum } (+S) = \frac{1}{2} p \frac{(l-x)^2}{l}$$

$$\text{Maximum } (-S) = \frac{1}{2} p \frac{x^2}{l}$$

as is well known. With a system of concentrated loads, the shear will be given by equation 13.

If we wish to ascertain whether the positive shear will be greater with $P_2$ at $c$ than with $P_1$ at $c$, we observe that if $P_2$ is moved up to $c$, $P_1$ will be moved forward a distance $a_1$ to the point $g$. If now the influence line were $B\ a'$, the shear would be increased by $Ra_1$ tan. $\alpha$, but since $P_1$ passes $c$ we must subtract $P_1 . (s\ t)$; hence the shear is really increased by $Ra_1$ tan. $\alpha - P_1\ (s\ t)$. But $s\ t = A\ a' = 1$; hence the shear is increased provided $R\ a_1$ tan. $\alpha > P_1\ (s\ t)$, or provided $R\ \frac{a_1}{l} > P_1$, as found on page 26. And here, as in the case of moments, we may conclude that whenever the influence line for any function is composed of two parallel straight lines connected by a vertical, the maximum value of the function will occur for the same loading which on a span $A\ B$ without floor-beams would cause the maximum shear at $c$.

*Shear; with Floor-Beams.*—A brief consideration will show that in this case the influence line for the shear in the panel $g\ h$ (Fig. 10, Plate I) is $B\ H\ G\ A$. When the load lies at $k$, the shear in the panel $g\ h$ is zero; when it lies to the right of $k$ the shear is positive, and when it lies to the

left of $k$ it is negative. The point $k$ is such a point that when a load lies there the reaction at $A$ will be equal to the load on the floor-beam at $g$, that is

$$P\frac{s}{l} = P\frac{z}{p} \text{ or } \frac{s}{l} = \frac{z}{p}.$$

This point $k$ may be called the neutral point of the panel $g\,h$, and each panel has such a neutral point. The influence line gives us the point $k$ directly, and shows that with a uniform load the maximum positive shear will occur when the load covers $B\,k$ alone, and the maximum negative shear when the load covers $A\,k$ alone. Calling $A$ the area $B\,H\,k$, and $A_2$ the area $A\,G\,k$, we have

$$\text{Maximum } (+S) = p\,A.$$
$$\text{Maximum } (-S) = -p\,A.$$

Since $H\,h = \frac{x^1}{l}$, and $G\,g = -\frac{x^2}{l}$, the lines $A\,G$ and $B\,H$ are readily seen to be parallel.

If the loading consists of a series of concentrated loads, equation 13 is to be used.

Finally, we may again deduce from the influence line the conditions determining the loading giving maximum shear for a series of concentrated loads. Starting with the first wheel at $h$, we must determine whether the shear is increased by bringing up $P_2$ to $h$. If in moving the loads in this way $P_1$ passes beyond $G$; or, in other words, if $a_1 > p$, we have the increase of shear equal to $Ra_1$ tan. $\alpha_1 - P_1\ (s\,t)$, or, since $s\,t = A\,a_1 = 1$, the shear will be increased if $R\frac{a_1}{l} > P_1$. Similarly, if $a_1 < p$, and $P_1$ moves to $s'\,t'$, the increase of shear will be $Ra_1$ tan. $\alpha_1 - P\ (s'\,t')$; or the shear will be increased if $R\frac{a_1}{l} > P\frac{a_1}{p}$ as already shown on page 26. We may again conclude, therefore, that whenever the influence line is composed of two parallel straight lines joined by an inclined line, as in Fig. 10, the loading causing the maximum value of the function will be the same loading that on a beam of length $A\,B$ would cause the maximum shear in the panel $g\,h$.

As a last illustration of influence lines, consider that for the stress in the bar $a\,b$ of the truss shown in Fig. 11, Plate I. Since the stress in $a\,b$ is simply the moment at $g$ divided by the height $h$, the influence line $A\,d\,B$ will be exactly similar to that for the moment at $g$, the ordinate $c\,d$ being $\frac{x\,(l-x)}{l\,h}$.

## IV. Example of Maximum Shears and Moments.

Although the calculation of maximum shears and moments is generally well understood by bridge engineers, the following example will illustrate the simplicity of the following methods, and will add to the completeness of this article.

In finding moments and shears, a diagram like that in Fig. 12, Plate II, should be used, as is done in several bridge offices. Fig. 12 is made out for a Pennsylvania Railroad consolidation engine, the lower line giving the loads on each pair of wheels; the next line above the distances of each load from the first load of six tons; the third line the sum of all the loads up to and including the load above which the figure occurs; and the upper line the sums of moments, about any load, of all the loads on its left. Thus, $6 \times 7.5 = 45$; $6 \times 12 + 12 \times 4.5 = 126$ (or more rapidly, $45 + 18 \times 4.5 = 126$). With the aid of this diagram, moments may be very rapidly calculated.

*Example.*—Let it be required to find the maximum stresses in the bars of the truss shown in Fig. 13, Plate I, for the loads in Fig. 12.

Chord Bars.—For the maximum chord stresses it is only necessary to find the maximum moments at the points $a$, $b$, $c$ and $d$.

*Maximum Moment at a.*—If we try the first driver just to the right of $a$, then if the locomotives are preceded, as well as followed, by a uniform load, as shown in Fig. 12, a portion of the panel $A\ a$ will be covered by this uniform load. We shall, however, not consider any load in front of the engines, as it is generally unnecessary in practice to do so, and ordinary specifications do not call for it. We then have the right-hand abutment $B$ 105 feet from the load 2, or 112.5 feet from 1, and therefore 10.5 feet beyond 18, the last 7.5 feet being therefore covered by the uniform load. To the left of $a$ we have six tons, to the right $166 + 7.5 \times 1.5 = 177.25$, and as there are seven panels to the right and one on the left, the average loads may be represented by $\frac{6}{1}$ on the left, and $\frac{177.25}{7} = 25 +$ on the right. As the average load on the right is greater, we must move the loads to the left. With the load 2 just to the left of $a$ we shall have the average loads,

$$\text{On the left, } \frac{18}{1} = 18; \text{ on the right, } \frac{165.25}{7} = 23 +;$$

hence we must still move to the left. With the load 3 just to the left of

$a$, we have $B$ 15 feet beyond the load 18, and 12 feet covered by the uniform load. The average loads are

$$\text{On the left, } \frac{30}{1} = 30; \text{ on the right, } \frac{160}{7} = 23 +.$$

Hence the system must be moved to the right, and therefore the load 3 must lie at $a$.

This case has been worked through in detail, but with practice the portion of the loads causing maximum moments may be so nearly assumed in the beginning that but one or two trials are necessary to determine it exactly. It must only be borne in mind that the entire span must be loaded as much as possible, with one heavy load at the point in question, and with the heaviest loads as near that point as possible. In the case just treated we should have tried the load 3 at first, if we had not desired to illustrate the method of procedure.

It must be remembered, however, that several distinct portions of the load may give maximum moments at the same point, and to decide between them the only exact way is to work out the actual moments. They will, however, in many cases be very nearly alike, and inspection will almost always show which will probably be the greater. Thus, in the above case, if we try the load numbered 8 on the right and left of $a$, we shall find that when at $a$ it gives a maximum moment; but in this case it is very evident that this second maximum will be smaller than the one corresponding to load 3 at $a$. In some cases, however, a trial may be necessary.

The maximum moment itself is easily found, the load 3 being at the point $a$. The reaction at $A$ is found by taking moments about $B$, which lies fifteen feet beyond the load 18. This moment is $8\,934 + 172 \times 15 + 18 \times 6 = 11\,622$, the last term $(18 \times 6)$ being the moment of the uniform load covering the last 12 feet of the span. The reaction is $\frac{11\,622}{120} = 96.85$ tons, and the moment at $a = 96.85 \times 15 - 126 = 1\,452.75$ foot tons. Finally, the stress in the bars $A\ a$ and $a\ b$ is $\frac{1\,452.75}{20} = 72.6$ tons for both trusses.

In using the schedule Fig. 12, care must be taken to allow for any of the front loads passing off the bridge, as will appear in what follows. The remaining moments may thus be calculated.

*Moment at b.*—In this case it will be found that the maximum moment occurs when load 5 is at $b$; but in all probability, since for this loading

the other drivers are all in front of $b$, a still greater moment will occur when the second engine reaches $b$. The trial shows, in fact, that a maximum occurs when load number 12 is at $b$. For this loading the span reaches from 37 to 157 on the line of distances, and we have

Reaction at left =

$$\frac{8\,934 + 172 \times 55 + 1\tfrac{1}{2} \times 52 \times 26 - 1\,327 - 70 \times 120.5}{120} = 88.83 \text{ tons.}$$

Moment at $b = 88.83 \times 30 - 126 - 8\,(20 + 25) = 2\,179$ foot tons.

$$\text{Stress} = \frac{2\,179}{20} = 108.95 \text{ tons (in } a'\,b' \text{ and } b\,c\text{).}$$

For the loading with load 5 at $b$ we have

$$\text{Reaction} = \frac{8\,934 + 172 \times 9 + 1\tfrac{1}{2} \times 6 \times 3}{120} = 87.575$$

Moment at $b = 87.575 \times 30 - 450 = 2\,177$, or almost precisely the same as the other, though a little smaller.

*Moment at c.*—A trial shows that load 13 gives maximum moment. The span reaches from 26.5 to 146.5 on the line of distances.

Reaction =

$$\frac{8\,934 + 172 \times 44.5 + 1\tfrac{1}{2} \times 41.5 \times 20.75 - 450 - 54 \times 125.5}{120} = 88.77$$

$$M = 88.77 \times 45 - 261 - 32 \times 32.25 = 2\,701.65.$$

$$\text{Stress in } b'\,c' \text{ and } c\,d = \frac{2\,701.65}{20} = 135.08 \text{ tons.}$$

*Moment at d.*—The load 12 lies at the point, the span reaching from 7 to 127, and with 22 feet loaded with the uniform load.

$$R = \frac{8\,934 + 172 \times 25 + 1\tfrac{1}{2} \times 22 \times 11 - 6 \times 127}{120} = 106.96$$

$$M = 106.96 \times 60 - 3\,948 - 6 \times 67 = 3\,321.6$$

$$\text{Stress} = \frac{3\,321.6}{20} = 166.08 \text{ (in } c'\,d'\text{).}$$

SHEARS.—*Panel A a.*—With $P_1$ at $a$, the total load on the span is 172 tons. Now since $172 \times \frac{7.5}{120}$ is greater than $6\,\frac{7.5}{15}$, the shear will be increased by bringing up $P_2$. If $P_3$ be brought up, $P_1$ will still remain on the bridge; the reaction will be increased by $183.25\,\frac{4.5}{120} + d$ (on account of the uniform load coming on, $d$ being exactly equal to $1\tfrac{1}{2} \times 4.5 \times \frac{2.25}{120}$), or neglecting $d$, by 6.87. But we must substract $\frac{4.5}{15}.18$ more than when $P_2$ is at $a$; hence since 6.87 is greater than this (or 5.4), the

shear will be still farther increased if $P_3$ is brought up to $a$. It could in the same way easily be found that if $P_4$ were advanced to $a$ the shear would be diminished, regard being had to the fact that here $P_1$ would pass off the bridge. The maximum shear, with $P_3$ at $a$, is found as follows, the span ending at 117 feet from $P_1$, and the last twelve feet being covered by the uniform load:

$$\text{Reaction at left} = \frac{9\,450 + 172 \times 12 + 18 \times 6}{120} = 96.85$$

$$\text{Shear} = 96.85 - \frac{12}{15}.6 - \frac{4.5}{15}.12 = 88.45 \text{ tons.}$$

*Panel a b.*—With $P_1$ at $b$, $R = 148$; and since $148 \frac{7.5}{120} > 6 \frac{7.5}{15}$ $\left[\text{or } 148 > 6 \frac{120}{15} (= 48)\right]$ the shear is increased by bringing up $P_2$. Again (the total load being now 164), since $164 \frac{4.5}{120} (= 6.15) > \frac{4.5}{15}.18$ $(= 5.4)$, it is still further increased by bringing up $P_3$. Finally, since $172 \frac{4.5}{120} (= 6.45) < \frac{3}{15}.6 \times \frac{4.5}{15}.24 (= 8.4)$, it is diminished by bringing up $P_4$. Hence $P_3$ must lie at $b$, and

$$\text{Maximum shear} = \frac{8\,934}{120} - \frac{12}{15}.6 - \frac{4.5}{15}.12 = 66.05$$

*Panel b c.*—With $P_1$ at $c$, the total load $R = 128$, and since $128 > 48$, we must bring up $P_2$. And since $140 \frac{4.5}{120} (= 5.25) < 5.4$ (as above), we must not bring up $P_3$. Hence $P_2$ must be at $c$, and

$$\text{Maximum shear} = \frac{5\,046 + 140 \times 6.5}{120} - \frac{7.5}{15}.6 - 45.88.$$

*Panel c d.*—With $P_1$ at $d$, $R = 92$, and since $92 > 48$, we must bring up $P_2$. It is not necessary to try $P_3$, since in the previous panel it was not advanced to $c$. With $P_2$ at $d$,

$$\text{Maximum shear} = \frac{3\,948 + 116 \times 0.5}{120} - \frac{7.5}{12}.6 = 29.63$$

*Panel d e.*—With $P_1$ at $e$, $R = 78$; and since $78 > 48$, we must advance $P_2$, and

$$\text{Maximum shear} = \frac{2\,102 + 86 \times 5.5}{120} - \frac{7.5}{15}.6 = 18.46$$

*Panel e f.*—With $P_1$ at $f$, $R = 54$, hence $P_2$ must be advanced.

$$\text{Maximum shear} = \frac{1\,327 + 70 \times 1}{120} - \frac{7.5}{15}.6 = 8.64$$

*Panel f g.*—With $P_1$ at $g$, $R = 30$, hence $P_1$ must be at $g$.

$$\text{Maximum shear} = \frac{126 + 30 \times 3}{120} = 1.8.$$

We have thus found the maximum positive shear in every panel.

## VI. Single Intersection Trusses.

Little more than the processes used in the example just given is necessary in the calculation of ordinary single intersection trusses with parallel chords. The chord stresses are found from the moments, the web stresses from the shears. In finding chord stresses in Warren girders however, it is necessary, in order to obtain the stresses in the loaded chord, to calculate maximum moments at the opposite joints of the unloaded chord, that is, at points between the floor-beams. In this case however, as already remarked, it is sufficient to calculate exactly as though there were no floor-beams present, the results being then a little too large; but, if desired, influence lines may be used, with the application of equation 13. For example, in Fig. 11, Plate I, to find the maximum stress in the bar $d$, it is necessary to find the maximum moment at the point $a$. The influence line for the stress in $d$ is given in Fig. 14; and calling $G_1$ the total load on $g\,B$, $G_2$ that on $h\,g$, and $G_3$ that on $A\,h$, then the stress in $d$ will be increased by moving the loads to the left, so long as

$$G_1 \tan.\ \alpha_1 > G_2 \tan.\ \alpha_2 + G_3 \tan.\ \alpha_3$$

or, substituting for tan. $\alpha_1$, tan $\alpha_2$, tan. $\alpha_3$, their values, so long as

$$G_1 > G_2 \frac{l-2\,t}{2\,t} + G_3 \frac{l-t}{t} \quad \text{or}$$

$$G_1 + G_2 + G_3 > G_2 \frac{l}{2\,t} + G_3 \frac{l}{t} \tag{14}$$

The position of the load giving maximum stress being found, use equation (13).

Further, it is necessary to consider the effect of counters on the use of influence lines. Thus, in Fig. 13, suppose that there is no counter $c\,d'$, the influence line for the diagonal $c'\,d$ will be $A\ C\ D\ B$ in Fig. 15. If, however, the counter be in existence, the true line will be $A\ F\ D\ B$. In calculating diagonals, however, by means of influence lines, it is clear that we must treat each one as though there were no other diagonal in the same panel, or using for $c'\,d$ the line $A\ C\ D\ B$. For, if the diagonal is in action, every load on the span affects it, causing either tension or compression, and if the resultant stress for any particular loading is

compression, it simply shows that the diagonal considered would for that loading not be in action, but that the other one would be; and if this last is treated in the same way, the load in question will be found to produce tension in it. Similarly, in calculating chords and verticals, the influence line must be drawn on the supposition that certain diagonals are in action. Then, for any particular loading, it must first be determined whether such diagonals are really in action or not, and if not, then in getting the chord or vertical stress for such loading, the influence line used must be one constructed on the basis of the diagonal which is really in action.

Let us now proceed to some applications of these principles.

### VI. Application to Floor-Beams, Floor-Beam Hangers, End Suspenders, etc.

In the truss shown in Fig. 13, the greatest load on any floor-beam, as $a$, occurs for some particular position of the loads, which should be determined, as upon it will depend the calculation of the floor-beam and its connections with the stringer, of the floor-beam hangers, and of the end suspender or hip vertical $a\ a'$. Fig. 16 represents the influence line for the load on the point $a$, the ordinate $n\ n'$ being unity. This line is exactly similar to that for the moment at $a$ on a beam of length $A\ b$; hence the loading which would cause the maximum moment in the latter case will also cause the maximum load on $a$. Hence we see that to cause the greatest stress on floor-beams, hangers or hip verticals, the two adjacent panels should be loaded in the same way as would give the maximum moment at the floor-beam considered on a span equal to the two adjacent panels. This simple theorem applies to all cases of posts or ties such as $a\ b$ in Figs. 17, 18, 19 and 20, Plate I.

Let us consider further the strut $c\ d$ in Fig. 20. The influence line is shown below, $n\ n'$ being unity, and it is therefore easily seen that for the maximum stress the loading should be such as would give the maximum moment at $c$ on a beam of length $A\ f$. The same is true regarding the tie $c\ d$ in Fig. 19.

For the strut $f\ g$ in Fig. 20, the influence line is shown below, $n\ n'$ being unity, and it follows that the loading giving maximum stress will be the same which would give the maximum moment at $f$ on a span $A\ B$.

Moreover, the simplest way of finding the stress is to find actually, by means of the diagram Fig. 12, the maximum moment for the corre-

sponding loading, and then to multiply it by the ratio which the ordinate of the influence line for the actual stress bears to the ordinate of the influence line for moment. For example, in Fig. 16 the ordinate $n\ n'$ of the line for stress is 1, while for moments it would be $\frac{15}{30}.15 = 7.5$. Hence the maximum stress in $a\ a'$ in tons will be equal to the maximum moment at $a$ on a beam $A\ b$, in foot tons divided by 7.5. Similarly, for $c\ d$ in Fig. 20, supposing the panels to be 20 feet long, the ordinate for stress is 1, and for moments $\frac{40}{80}\ .40 = 20$; hence

$$\text{Maximum stress in } c\ d \text{ (tons)} = \frac{1}{20} \text{(max. moment at } c \text{ in foot tons).}$$

## VII. Fink and Bollman Trusses.

These trusses are taken up here as offering some analogy to cases just discussed. The loading causing maximum stresses in the verticals has already been fully explained, together with the method of determining such stresses. The stresses in diagonals follow at once from those in the verticals, and need not be further discussed. In considering the chords, an interesting application of influence lines may be made.

Fig. 21, Plate II, represents a Fink truss in which we have to find the maximum chord stresses. The influence line for the bar $a$ is the one in full lines in Fig. 21A, and the ordinates may be easily checked. From this the loading causing maximum stress may be easily found by trial, using equation 13, but a rule may also be deduced which will save time.

Let $G_1$ = resultant load on $ei$.
$G_2$ = " " $ce$.
$G_3$ = " " $bc$.
$G_4$ = " " $ab$.

Starting with any given position of the loads, if the system be moved to the left a distance $d\,x$, it will be easily seen that the stress will be increased if

$$G_1 \text{ tan. } \alpha_1 > G_3 \text{ tan. } \alpha_2 + G_4 \text{ tan. } \alpha_3; \quad \text{or if}$$

$$G_1 > G_3 + 3\ G_4 \tag{15}$$

By the aid of this condition, the loading causing maximum stress may be readily found, and then the actual stress by equation 13.

The influence line for the bars $b$ is the one dotted in Fig. 21A. In this case, if we call $G_1$, $G_2$, $G_3$, the total loads on $d\,i$, $c\,d$ and $a\,c$ respect-

ively, the loading causing maximum stress will be found by moving to the left if

$$G_1 > G_2 + 2\,G_3 \tag{16}$$

In the Bollmann truss shown on Fig. 22, Plate II, the stress in the upper chord is constant throughout, and the influence line is given in Fig. 22A. The vertices all lie on a parabola. If the loads on each panel are called $G_1, G_2, G_3, G_4, G_5, G_6, G_7, G_8$, we may find the loading causing maximum stress by the condition that the loads must be moved to the left, provided

$$7G_1 + 5G_2 + 3G_3 + G_4 > G_5 + 3G_6 + 5G_7 + 7G_8 \tag{17}$$

## VIII. The Truss with Intermediate Secondary Systems.

Under this head we include trusses like those shown on Figs. 23, 24 and 25.

In Fig. 23, Plate II, the stress in $a$ will be obtained by finding the maximum moment at $B$ and dividing by $h$; that in $b$ by finding the maximum shear in the panel $D\,B$; that in $c$ as already explained. The vertical component of that in $d$ will be one-half that in $c$; for in Fig. 23c we have at the joint $K$ the equations of equilibrium as follows:

$$S_1 \sin.\ \alpha + S_4 \sin.\ \alpha - S_2 \sin.\ \alpha = o \tag{18}$$

$$S \cos.\ \alpha - S_4 \cos.\ \alpha - S_2 \cos.\ \alpha - S_3 = o \tag{19}$$

from which we obtain

$$S_4 = -\frac{1}{2} S_3 \sec.\ \alpha \tag{20}$$

$$S_1 = S_2 + \frac{1}{2} S_3 \sec.\ \alpha \tag{21}$$

To find the maximum stress in $e$, draw its influence line, as in Fig. 23A, which is drawn for the vertical component of the stress in $e$, regardless whether there be a counter, as in the panel to the right. It will be found that this influence line is exactly the same as it would be if there were no bars $c$ and $d$. The ordinates at $B'$ and $C'$ are clearly $\frac{s}{l}$ and $\frac{s'}{l}$ respectively, and if $G$ and $H$ be joined by a straight line, the ordinate to this line at $D'$ will be $\frac{s}{l} - \frac{1}{2}\frac{(s+s')}{l} = \frac{1}{2}\frac{s-s'}{l}$; but this can be shown to be the ordinate to the influence line at $D'$ by considering that if a load of unity lies at $D$, the shear in the panel $CD$ is $\frac{s + \frac{l-s-s'}{2}}{l}$ while

the shear in the bar $d$ is, according to equation 20, a compression of $\frac{1}{2}$, hence the shear in the bar $e$ is

$$\frac{s + \frac{l - s - s'}{2}}{l} - \frac{1}{2} = \frac{s - s'}{2\,l}$$

Hence the loading causing the maximum stress in $e$, as well as that maximum stress itself, is found entirely regardless of the secondary systems, just as though the truss were an ordinary Pratt truss with joints only at $C$, $B$, etc.

The stress in $f$ is the same as that in $g$. To find it, draw the influence line, as in Fig. 23B. $A_1\ C\ B\ B_1$ is the line we should have if there were no secondary system, and the ordinates at $C'$ and $B'$ must clearly be the same in the present case. We have only to find the ordinate at $D'$, since between any two consecutive joints the line must always be straight. With a load unity at $D$, we find the stress in $f$ by taking moments about $O$, allowing for the moment of the stress in $d$. Thus,

$$\text{Reaction on the left} = \frac{s + \frac{1}{2}p}{l} \qquad (p = \text{panel length}).$$

$$\text{Hence component of stress in } d = \frac{1}{2}\,\frac{p}{h} \qquad \therefore$$

$$\frac{s + \frac{1}{2}p}{l} \cdot s' + \frac{1}{2}\frac{p}{h} \cdot h = g\,h \qquad \therefore$$

$$g = \frac{s\,s'}{l\,h} + \frac{s'\,p}{2\,l\,h} + \frac{p}{2\,h} \tag{22}$$

The ordinate to $C\ B$ at the point $D'$ would be

$$y = \frac{s\,s'}{l\,h} + \frac{s'\,p}{2\,l\,h}$$

hence the point $D$ is above the line $C\,B$ by the distance $\frac{p}{2\,h}$. In other words, the stress in $g$ and $f$ is equal to that which would exist if there were no secondary system, plus the stress in $C\,B$ considered as the chord of the secondary truss $C\,K\,B$. The ordinate at $D'$ will be the longest in the curve, even greater than $C\ C'$ in every case. The loading causing maximum stress will be nearly the same as if there were no secondary system, but there will generally be a load at $D$.

The stress in the vertical, $h$, is readily found, being the vertical component of that in the diagonal $e$, plus whatever load may be on $O$. Since the stress in $e$ is the same as without the secondary system, the stresses in the verticals will be also. The same is true of the counters.

Fig. 24, Plate II, represents a deck bridge of the same system as Fig. 23, and the same laws hold as for the bars bearing the same letters in Fig. 23.

Fig. 25, Plate III, represents a triangular truss with three intermediate verticals in each long panel $A\,E$. The stresses in this truss may be found as follows:

That in $a$ equals the maximum moment at $A$ divided by $b$, the loading being the same as would cause the maximum moment at $A$ on a simple beam of the same span.

Those in $b$, $b'$, and $d$ are found as has been already explained.

That in $c$ and $c'$ is equal to one-half of that in $b$, or $b'$, resolved along the bars.

That in $e$ has for its vertical component the maximum shear in the small panel $A\,B$, the beam being treated as a simple beam with panels equal to $A\,B$.

That in $e^1$ is found in a similar way, from the shear in $D\,E$.

For $f'$ the influence line is drawn in Fig. 25A, showing that the stress is just the same as in a truss with panels equal to $A\,C$ and $C\,E$, that is, disregarding the small systems $A\,G\,C$, $C\,K\,E$, etc. The same is true of $f$.

For $g$ and $h$ the stress will evidently be the same as in the bar $A\,C$ of a truss with panels equal to $A\,C$, $C\,E$, etc., plus the stress in $A\,C$ as the chord of the small truss $A\,G\,C$. The influence line may be easily drawn and the maximum loading will, as in Fig. 23, generally be with some load at $B$. Similar remarks apply to the bars $k$, $l$.

## IX. Trusses with Multiple Systems of Web.

Trusses belonging to this class—double intersection trusses, as they are commonly built in this country—have always offered some difficulties in calculation for the actual loads. If a uniform load be assumed, with locomotive excess, the calculation is very simple, for the load coming on each of the systems of which the frame consists is definitely determined. When it comes, however, to consider the actual loads, it is by ordinary methods not easy to tell just what loading causes the maximum stresses; although, having that loading, the stresses themselves may be found without difficulty. The writer believes that the calculation with locomotive excesses is amply sufficient for all practical purposes, and he believes that, in view of the rapid increase of the weight of engines and

cars, the uniform load should be rather larger than generally taken, say 3 300 or 3 500 pounds per foot. The assumption of one particular engine with its actual load, which in a few years may be exceeded in weight by other engines on the same road, seems to render the assumption of actual loads for other cases than the calculation of the floor, of hip verticals, etc., rather a needless refinement, and in some cases quite as inaccurate as the assumption of a reasonable uniform load with locomotive excesses would be. Nevertheless, actual loads are used in many large offices, even in the calculation of double intersection trusses, and the following methods may therefore prove acceptable to some. The use of influence lines will be found to afford some aid in such cases.

Let us consider, for instance, the ordinary Whipple truss shown in Fig. 26, Plate III. We have seen that the lines of influence enable us to find: (1) the points where a single load must be to cause no stress in the bar considered; (2) the portion of the structure which must be covered by a uniform load in order to cause the maximum positive or negative stress; (3) the value of such stress, by means of areas; (4) for concentrated loads the loading causing maximum stress of either kind, by trial, or by deducing simple rules; (5) the value of such stress by the use of equation 13.

In Fig. 26 we find the influence line for the chord piece $a$ to be the broken line shown in Fig. 26A, the ordinates being indicated in the figure. There is some uncertainty regarding the loads on the joints 1 and 15. Thus, if we suppose the $\frac{1}{15}$ which goes to the left from joint 1 to go through the heavy system, the stress in $a$ is $\frac{6}{16}\frac{p}{h}$, while if it goes through the light system it is $\frac{5}{16}\frac{p}{h}$, and if half on each it is $\frac{11}{32}\frac{p}{h}$. Overlooking this very small uncertainty, the apices of the influence line lie alternately on lines radiating from $B$ and $A$. The numbers on the ordinate in the figure are, of course, to be multiplied by $\frac{p}{16\,h}$. Thus the stress in $a$ for a load unity at 11 is $\frac{55}{16}\frac{p}{h}$. These ordinates may be easily found by any of the ordinary methods of calculation. Thus, of a load on 11, $\frac{11}{16}$ goes to the left abutment, and is the vertical component in the end post and all the diagonals of the light system to the left of $a$. The stress on the first and second upper chord pieces is therefore $\frac{11}{16}\frac{p}{h} + \frac{11}{16}\frac{2\,p}{h}$; and on the third and fourth

$$\frac{11}{16}\frac{p}{h} + \frac{11}{16}\frac{2\,p}{h} + \frac{11}{16}\frac{2\,p}{h} = \frac{55}{16}\frac{p}{h}.$$

To find the loading causing the maximum stress in $a$, it is evident that the heaviest loads must be as near the point 10 as possible, with probably one load at that point. The skeleton of the truss and the influence line being drawn to scale, and the loads being plotted on tracing-cloth to the same scale, the loading causing maximum stress may be found by trial, placing the loads with one of the heaviest at 10, and finding $\Sigma\, P\, y$, then trying another load at 10 until the maximum is found. Or the following method may be used: first write on each panel the tangent of the angle which the influence line in that panel makes with the horizontal, calling it positive when the ordinate increases in going toward the left. This has been done in Fig. 26A on the line $A\ B$, the numbers to be multiplied by $\frac{p}{16\,h}$ to obtain the actual tangents. Now place the tracing, with the plotted loads, over the truss, with one of the drivers just to the right of the point 10, and call $G_1$, $G_2$, $G_3$, etc., the total loads in the successive panels, commencing at the right. These loads can be very rapidly added up, and written above the numbers giving tangents. Then the stress will be increased by moving the system of loads to the left, provided

$$\Sigma\ (G \tan.\ \alpha) > 1 \qquad (23)$$

or in this case, if

$$6\ G_1 + 6\ G_2 + 3\ G_3 + 9\ G_4 + G_5 + 11\ G_6 - G_7 + 13\ G\ - 3\ G_9 + 15\ G_{10} - 5\ G_{11} - 15\ G_{12} - 7\ G_{13} - 13\ G_{14} - 9\ G_{15} - 11\ G_{16} > 1 \quad (24).$$

Should this inequality hold, then we move the system to the left until the load which was previously just to the right of 10 is just to the left of the same point, and again apply the test. Should the inequality be now reversed, it would follow that the load in question must lie at the point 10 in order that the stress should be a maximum, and the stress itself would be obtained by equation 13. With a little practice this method will be found more rapid than appears at first sight, and it is of course as accurate as any method can be. All the bars of the truss may be treated in the same way, and it is not necessary to refer to them further. Fig. 26B, however, gives the influence line for the diagonal $b$, showing the general shape of all the lines for the diagonals.

Triangular trusses with two, three or more systems of web, and in fact all multiple systems, may be treated in the same way, and offer ground for no special remark.

## X. Trusses with Curved Chords.

A. *Stresses in the Diagonals.*—Taking the most common case of a truss with curved chords, that shown in Fig. 27, Plate III, we shall first have to investigate the loading causing maximum stresses in diagonals. Considering the diagonal $b$, the ordinates to the influence line may best be obtained by the method of moments, the section being $s\,s'$, and the origin of moments $O$, at a distance $m$ to the left of the abutment $A$. The lever arm of the bar $b$ is $s$, as shown, and the ordinates at $C$ and $D$ may be easily checked. If we have to do with a uniform load, we have now only to find the areas of the two triangles $A_1$ and $A_2$, and if $p$ is the live and $g$ the dead load per running foot, the maximum tension in $b$ will be

$$(p + g)\,A_1 - g\,A_2 \tag{25}$$

and the minimum tension will be

$$g\,A_1 - (p + g)\,A_2 \tag{26}$$

and if the latter comes out negative it shows compression, or the necessity for a counter.

If we are dealing with a system of concentrated loads, the loading causing maximum stress may be found as follows: As the train approaches from the right, the stress will evidently increase until the first wheel reaches $D$, since, when there are no loads to the left of $D$, the stress is simply equal to $\frac{R_1\,m}{s}$. In general, if $M$ be the moment, about the point $O$, of all the loads and reactions to the left of $D$, the stress in the bar will be $S = \frac{M}{s}$.

Now if the first wheel be advanced beyond $D$, the reaction will be increased, and therefore its moment, but there will be a load on $C$, whose moment is to be subtracted, and both of these changes will take place uniformly as the load advances to the left. The question therefore is, as before, whether the stress will be greater with the first or with the second load at $D$. Let $R =$ the total load on the bridge when the first wheel is at $D$; then if the loads be all advanced a distance $a_1$, bringing the second wheel to $D$, the reaction will be increased by $\left(R\,\frac{a_1}{l} + d\right)$, and its moment therefore by $\left(R\,\frac{a_1}{l} + d\right) m$.

But we shall now have to subtract the moment of the load on $C$ and all joints to the left. Let the distance $C\,O = n$, and if $a_1 > p$, let the distance from $O$ to $P_1$ when $P_2$ is at $D$, be called $r$. Then we must

subtract $P_1 \frac{a_1}{p}.n$ or $P_1\ r$, according as $a_1$ is less or greater than $p$. We shall therefore increase the stress by moving the load to the left, provided that

$$\left(R\frac{a_1}{l}+d\right)m > P_1\frac{a_1}{p}.\ n \quad (\text{when } a_1 < p) \tag{27}$$

$$\left(R\frac{a_1}{l}+d\right)m > P_1\ r \qquad (\text{when } a > p) \tag{28}$$

Since we can almost always neglect $d$, which is the reaction due to any load which may come on the span from the right during the operation, we have generally

$$R\frac{a_1}{l}m > P_1\ \frac{a_1}{p}n, \text{ or } R\frac{m}{l} > P_1\ \frac{n}{p} \text{ (when } a_1 < p) \tag{29}$$

$$R\frac{a_1}{l}\ m > P_1\ r, \text{ or } R\frac{m}{l} > P_1\frac{r}{a_1} \qquad (\text{when } a_1 > p) \tag{30}$$

Similar rules may easily be obtained with reference to the question of moving the third load to $D$, but they are not often necessary, and the method will be clear from the example below.

An example will show how easy these rules are of application. In Fig. 28, Plate III, let us consider the bar $b$. We first find $m$ by the proportion

$$m + 40 : m + 60 :: 24 : 30$$
$$m = 40 \text{ feet.}$$

With the loads shown in Fig. 12 the stress in $b$ will therefore be increased by moving up the second wheel to $D$, since

$$164\ \frac{7.5}{160}.\ 40 > 6.\ \frac{7.5}{20}.\ 80$$
$$41 > 24$$

Further, the stress would be still further increased by moving up the third wheel, provided

$$176\ \frac{4.5}{160}.\ 40 > 18.\frac{4.5}{20}.\ 80$$
$$447 > 72$$

hence the third load should not be moved up.

For counters, we may consider the diagonals sloping in the same direction from one end of the bridge to the other, thus calculating both main diagonals and counters. Thus in Fig. 29 we consider the truss to be as shown, and calculate all the diagonals. In the case of $b'$, for instance, which is a counter, we proceed as follows:

The origin of moments is at $O$, the lever arm of $b'$ is $s'$, and the

influence line is drawn in Fig. 29A. In this case the stress will be greater when $P_2$ is at $D$ than when $P_1$ is at $D$, provided

$$\left(R' \frac{a_1}{l} + d\right) m' > P_1 \frac{a_1}{p} n' \text{ (when } a_1 < p) \tag{27A}$$

$$\left(R' \frac{a_1}{l} + d\right) m' > P_1 \; r' \quad \text{(when } a_1 > p) \tag{28A}$$

or, neglecting $d$, when

$$R' \frac{m'}{l} > P_1 \frac{n'}{p} \quad (a_1 < p) \tag{29A}$$

$$R' \frac{m'}{l} > P_1 \frac{r'}{a_1} \quad (a_1 > p) \tag{30A}$$

just as before. $R'$ being the resultant load on the span when $P$ is at $D$, and $n'$ and $r'$ corresponding to $n$ and $r$.

For finding the actual stress in a diagonal, the ordinary method is the following (Fig. 30): Find the shear $V$ in the panel $C\ D$, and the stress $S_1$ in the upper chord. Then, since at the section $S\ S^1$ the sum of all the vertical forces must equal zero, we must have

$$V - S_1 \sin. \alpha_1 - S_3 \cos. \alpha_3 = o$$

$$S_3 = \frac{V - S_1 \sin. \alpha_1}{\cos. \alpha_3}$$

or, in other terms, we subtract from $V$ the vertical component of $S_1$ to obtain the vertical component of $S_3$. The following method, however, will be found simpler and more expeditious. Let $M_1$ and $M_2$ be the moments, and $h_1$ and $h_2$ the heights of the truss at the points $D$ and $C$ respectively. Now, at the section $S\ S^1$ we must have the sum of the horizontal forces equal to zero; hence

$$S_1 \cos. \alpha_1 - S_2 - S_3 \sin. \alpha_3 = o \tag{31}$$

but since $S_1 \cos. \alpha_1 = \frac{M_1}{h_1}$, and $S_2 = \frac{M_2}{h_2}$

this reduces to $S_3 = \left(\frac{M_1}{h_1} - \frac{M_2}{h_2}\right) \text{cosec. } \alpha_3$ (32)

Take, for instance, the bar $b$ in Fig. 27. The reaction at the left, for the loading causing maximum tension in $b$, is found to be 61.78 tons, and the load on $C$ is 2.25 tons. Then we should have by the usual method

$$\frac{61.78 \times 60 - 2.25 \times 20}{30} = 122.06 = S_1 \cos. \alpha_1$$

$$122.06 \times \frac{6}{20} = 36.62 = S_1 \sin. \alpha_1$$

$$(61.78 - 2.25 - 36.62) \frac{31.24}{24} = 29.78 = S_3$$

By the last method we should have

$$\frac{61.78 \times 60 - 2.25 \times 20}{30} = 122.06 = \frac{M_1}{h_1}$$

$$\frac{61.78 \times 40}{24} = 102.97 = \frac{M_2}{h_2}$$

$$(122.06 - 102.97)\,\frac{31.24}{20} = 29.82 = S_3$$

B. *Stresses in the Verticals.*—These are best found by the simple method of moments. Thus in Fig. 30 the maximum stress in the vertical at $C$ would occur for a load up to $D$, the origin of moments being at $O$. The loading causing maximum stress may be found just as has been explained, and the stress itself by taking moments about $O$.

C. *Stresses in the Chords.*—These also are found by the method of moments, finding the maximum moment at the proper joint opposite the chord piece in question, and dividing by the lever arm. Where there are counters it is necessary to make sure which diagon is in action in the panel in question, so that the correct origin of moments may be taken. For example, in Fig. 30, if the main diagonal $C^1\ D$ is in action, the origin of moments for the upper chord piece $C^1\ D^1$ is at $D$, and for the lower chord piece $C\ D$ it is at $C^1$, while if the counter is in action the origin for the upper chord is at $C$, and for the lower at $D^1$. In order to find maximum chord stress, it is therefore necessary to find the moments as follows:

$$\left\{\begin{array}{l}\text{Maximum } M \text{ at } B\\ \text{Corresponding } M \text{ at } C \text{ (\textit{i.e.}, for the same loading which causes}\\ \quad\text{the maximum at } B)\end{array}\right.$$

$$\left\{\begin{array}{l}\text{Maximum at } C\\ \text{Corresponding } M \text{ at } B\\ \text{Corresponding } M \text{ at } D\end{array}\right.$$

$$\left\{\begin{array}{l}\text{Maximum at } D\\ \text{Corresponding } M \text{ at } C\\ \text{Corresponding } M \text{ at } E\end{array}\right.$$

$$\left\{\begin{array}{l}\text{Maximum } M \text{ at } E\\ \text{Corresponding } M \text{ at } D\\ \text{Corresponding } M \text{ at } F\end{array}\right.$$

Now, since in any panel, such as $C\ D$, the diagonal $C^1\ D$ is in action, provided $\frac{M_1}{h_1} > \frac{M_2}{h_2}$, as seen from equation (32), the above moments,

divided by the corresponding heights, enable us to determine easily the maximum stresses in the chords.

Suppose for instance that

$$A \left\{ \begin{array}{ll} \text{Maximum} & M \text{ at } C = 480: \dfrac{M_c}{h_c} = 20 \\ \text{Corresponding } M \text{ at } D = 570: \dfrac{M_d}{h_d} = 19 \end{array} \right\}$$

$$B \left\{ \begin{array}{ll} \text{Maximum} & M \text{ at } D = 630: \dfrac{M_d}{h_d} = 21 \\ \text{Corresponding } M \text{ at } C = 432: \dfrac{M_c}{h_c} = 18 \end{array} \right\}$$

From this it is evident that when the maximum $M$ at $C$ occurs, the diagonal $C D^1$ (counter) is in action, since $\dfrac{M_d}{h_d} < \dfrac{M_c}{h_c}$

Hence in this case

$$\left\{ \begin{array}{l} \text{Horizontal component of stress in upper chord} = 20 \\ \text{Horizontal component of stress in lower chord} = 17 \end{array} \right\}$$

When the maximum $M$ at $D$ occurs, the diagonal $C^1 D$ is in action, since $\dfrac{M_d}{h_d} > \dfrac{M_c}{h_c}$. Hence

$$\left\{ \begin{array}{l} \text{Horizontal component of stress in upper chord} = 21 \\ \text{Horizontal component of stress in lower chord} = 18 \end{array} \right\}$$

Therefore we have

$$\left\{ \begin{array}{l} \text{Horizontal component of maximum stress in upper chord} = 21 \\ \text{Horizontal component of maximum stress in lower chord} = 19 \end{array} \right\}$$

It will be easily seen that the stress in the upper chord is obtained from the greatest moment at either of the opposite joints, while that of the lower chord is obtained from the greatest of the two smaller moments in the sets $A$ and $B$ (19 and 18 in the above case).

## XI. Cantilevers.

The stresses in cantilevers may be in many cases most easily found by the use of influence lines, and many of the principles which we have demonstrated find apt application in bridges of this class. We will therefore consider such structures somewhat in detail. As ordinarily built, for three spans, cantilevers are of two kinds: (1) those in which the cantilever extends over the center span into the side spans, support-

ing a simple girder on each end; and (2) those in which two cantilevers extend over the side spans and out into the center span, supporting a simple girder between them. Let us consider first the former kind, represented in Fig. 31, Plate IV. In both cases it may be premised, however, that the simple girders are to be treated exactly as any simple girder would be, so that we have here to do only with the cantilever portions.

(*a*) *Stresses in the Diagonals.*—In the truss represented in the figure, which has horizontal chords and a single web system, the web stresses are obtained directly from the shears, and the influence lines are drawn simply for the shears, or vertical components of the web stresses. Were the chords inclined, the influence lines would be drawn for the actual stresses in the bars, which could easily be found by the method of moments. Fig. 31A represents the influence line for the bar $a$. Any load on the span $E\ F$ is supported partly at $E$, and the supporting force at $E$ increases directly with the distance of the load from $F$. A load unity at $E$ causes a downward reaction of $\frac{5}{12}$ at $C$, and this is the shear in $a$. The ordinate at $G$ is $\frac{8}{12}$, that at $H$, $\frac{4}{12}$, and that at $B$, $\frac{5}{12}$. Hence $B'\ H'$ and $G'\ E'$ are parallel straight lines, and the same is true of $a'\ B'$ and $E'\ F'$. For the maximum tension in $a$, the span $A\ C$ must be loaded, and also the span $C\ D$ from $D$ to some point between $H$ and $G$. For the maximum compression in $a$ (or the maximum tension in the counter) the remainder of the bridge must be loaded. If the load is uniform we have simply to find the areas $A_1$, $A_2$, $A_3$, $A_4$, and we have, if $g$ is the dead and $p$ the live load per running foot,

$$\text{Maximum tension in } a = (g + p)\ (A_1 + A_3) - g\ (A_2 + A_4) \qquad (33)$$

$$\text{Maximum compression in } a = (g + p)\ (A_2 + A_4) - g\ (A_1 + A_3) \qquad (34)$$

If the actual concentrated loads are to be considered, for the maximum tension $D'\ K'$ must be heavily loaded, with the heaviest loads nearest to $G$, and $A'\ C'$ must be also fully fully loaded, while $C'\ K'$ must be entirely unloaded. I do not know that there is any uniform practice in this country with regard to cases like the above, where the loaded portions of a bridge are separated by portions entirely unloaded. In determining the maximum stresses in continuous girders, alternate spans must be loaded and unloaded, and in Austria it is fixed by law that the load may be divided into two parts, but no more. In the above case it will no doubt be sufficient to consider the actual loads to cover one portion ($A'\ C'$ or $K'\ D'$), and the uniform load the other, in

whichever way will give the greatest stress. If we therefore place the concentrated loads on $A\ C$, we must place them in the same way as to cause the maximum moment at $B$, and find that moment $M$. Then, according to what has been said on page 48, the stress in the bar $a$ (in tons) due to the live load on $A'\ C'$ will be $\frac{1}{160}$ of the moment $M$ (in foot tons), since for a load unity at $B$ the moment at $B$ would be $\frac{100 \times 200}{300}$ $= 66\frac{2}{3}$, while the stress in $a$ would be $\frac{5}{12}$, and therefore

$$\text{Stress} : M :: \frac{5}{12} : 66\tfrac{2}{3}$$

$$\text{Stress} = \frac{5}{12}\ \frac{3}{200} M = \frac{1}{160} M$$

Then the vertical component of the total maximum tension in $a$ (in tons) would be ($g$ and $p$ being in tons per running foot)

$$V = g\ A_1 + (g + p)\ A_3 - g\ (A_2 + A_4) + \frac{1}{160} M \qquad (35)$$

We might then try the concentrated loads on $K'\ D'$ and the uniform load on $A'\ C'$, and see whether we obtain a greater stress.

The same method would be applicable to all the diagonals, counters and verticals in the span $C\ D$.

In the projecting portion $D\ E$, the influence line for the shear in the bar $b$ is shown in Fig. 31B, the ordinates at $E'$ and $L'$ being unity. In order to find the loadings giving the maximum stress, a simple rule may be deduced; thus let $G_1$, $G_2$, $G_3$ be the total loads on $E\ F$, $F\ L$ and $K\ L$ respectively. Then if the system of loads be moved a distance $d\ x$ to the left, the shear in $b$ will be increased, provided

$$G_1 \text{ tan. } \alpha_1, > G_3 \text{ tan. } \alpha_3 \qquad (36)$$

or if

$$\frac{G_1}{E\ F} > \frac{G_3}{K\ L} \qquad (37)$$

The actual stress may be found by methods already explained.

(*b*) *Stresses in the Chords.*—Fig. 31C shows the influence line for the stress in the chord piece $a'$. Two influence lines may be plotted for each chord piece in panels where there are counters, one on the supposition that the counter is in action, the other on the supposition that the main diagonal is in action, for the origin of moments will be different in the two cases. In order to find the stress due to any given loading, we must first determine which diagonal is in action under that loading, and then use the corresponding line for the chords. The full line is for the case when the main diagonal $G\ T$ is in action, the dotted

line for the case when the counter $H S$ is in action. It is evident that for the maximum tension in $a'$, the two outer spans must be completely loaded, and the center span entirely unloaded; further, that the heaviest loads must be in the span $A C$, and as near the point $B$ as possible, since the ordinate there is larger than anywhere else. We must therefore place the concentrated loads on $A C$, and the uniform load on $D F$. On $A C$ we find the loading causing maximum moment at $B$, and that moment $M$, and we then determine, from the influence line for the diagonals, whether for this loading the main diagonal or the counter is in action. In this case it will probably be the main diagonal $F G$, and, supposing this to be the case, we use the full line and proceed as follows: The ordinate at $B$ is $\frac{40\,p}{12\,h}$ but if the line $A'\ B'\ C'$ were for moments at $B$, the ordinate at the point would be $\frac{100 \times 200}{300} = 66\frac{2}{3}$, hence the stress in $a'$ due to the load on $A\ C$ will be

$$\frac{M\frac{40\,p}{12\,h}}{66\frac{2}{3}} = M\frac{p}{20\,h} \text{ (tons)} \qquad [M \text{ in foot tons}]$$

We therefore have for the maximum tension in $a'$

$$T \text{ maximum} = g\,A_1 - g\,A_2 + (g + p)\,A_3 + \frac{M\,p}{20\,h} \tag{38}$$

Similarly, if the line $C'\ G'\ D'$ were for moments, the ordinate at $G'$ would be $\frac{32}{12}\,p$, and if $M_1$ is the maximum moment at $G'$, the stress in $a'$ is $\frac{M_1}{h}$. Supposing that for this loading the main diagonal $T G$ is in action, the maximum compression in $a'$ is

$$C \text{ maximum} = g\,(A_2 - A_1 - A_3) + \frac{M_1}{h} \tag{39}$$

All the chord bars may be readily treated in this manner. Fig. 31A shows the influence line for the bar $b'$, in the projecting portion $D E$, in which portion no counters are necessary. If this line were for moments at $E$, the ordinate at that point would be $\frac{30}{13}\,p$. Hence if $M_2$ is the maximum moment at $E$ on a span $L F$, the maximum tension in $b'$ will be

$$T' \text{ maximum} = M_2\frac{3\,p}{h} \cdot \frac{13}{30\,p} = 1.3\,\frac{M_2}{h} \tag{40}$$

There is no compression in this bar.

Let us finally consider the cantilever shown in Fig. 32, Plate IV. The projecting portion $B\,C$ is identical with the portion $D\,E$ of Fig. 31, and the influence lines in Figs. 32A and B are the same as in Figs. 31B and D. Fig. 32C shows the influence line for the diagonal $a$, the lines $A'\,H'$ and $G'\,C'$ being parallel. The concentrated loads would have to be placed on $B\,D$ in the same way as would cause the maximum moment at $C$, and on $A\,G$ in the same way as would cause the maximum shear in the panel $H\,G$. We may therefore, as before, try the concentrated loads over either of these portions and the uniform load over the other, and take whichever gives the maximum stress. Fig. 32D gives the two influence lines for the chord bar $a'$, according as the main diagonal or the counter is in action, and the remarks on this matter made on page 49 apply here.

These illustrations are sufficient to show the application of this method to cantilevers, and will, I think, convince any one that when the actual loads are considered, it affords the easiest solution.

## XII. Continuous Girders, etc.

As these structures are seldom built in this country except for draw-bridges, it is not necessary to enter into a full and complete discussion of the calculation of stress by the use of influence lines. For concentrated loads, however, it will be found that the method is very convenient. Probably the simplest procedure is to find the reactions for a load unity at different points, and thus draw first the influence line for each reaction. This will enable the influence line for the stress in each bar to be constructed, after which the loading causing maximum stresses, and the stresses themselves, may be easily obtained.

Similar remarks will apply in the case of structures such as arches, with or without hinges. For the three-hinged arch, which is statically determined, this method is by far the easiest of application, even when uniform loads alone are considered. In this case it may be shown that the influence line for any bar consists of either three or four straight lines, so that the ordinates have to be found at only two, or at most three, points. The lines may be constructed rapidly, as the author knows by experience, and the areas are easily found. For concentrated loads there are some cases where it is difficult to see how the correct maximum stresses could be otherwise determined.

Were all the possible examples to be considered, the limits of this

paper would be unduly extended. Enough has been said, however, to bring to the attention of the profession in America this beautiful and fertile method.

## XIII. Literature.

In closing, a list of the works and papers treating on this subject, so far as the writer is acquainted with them, is appended, that those who desire may investigate the matter more in detail. The list is arranged chronologically, with some brief notes.

1. Weyrauch. "Allgemeine Theorie und Berechnung der continuirlichen und einfachen Träger," pp. 50 and 101. 1873. (First suggestion of influence lines applied to continuous girders, but method not developed.)
2. Fränkel. "Über die ungünstigste Einstellung eines Systems von Einsellasten auf Fachwerkträger, mit Hülfe von Influenzcurven." *Der Civilingenieur*, 1876. (Method quite fully developed.)
3. Winkler. "Beitrag zur Theorie der Bogenträger." Zeitschrift des Architekten und Ingenieurs Vereins zu Hannover. 1879. (Some remarks on influence lines applied to arches.)
4. Melan. "Beitrag zur graphischen Behandlung der Fachwerksträger, mit Zugrundelegung des Principes der Influenzcurven." Zeitschrift des Architekten und Ingenieurs Vereins zu Hanover, 1880, p. 221. (Application to parallel and curved-chord bridges, with some interesting results.)
5. Stelzel. "Theorie einfacher, statische bestimmter Brückenträger." Wien, 1880. (Application to shears and moments on a simple beam.)
6. Müller—Breslau. "Theorie und Berechnung der eisernen Bogenbrücken." Berlin, 1880. (Application to arches.)
7. Winkler. "Theorie der gegliederten Balkenträger." Wien, 1881. Second edition. (First edition, 1875.)
8. "Handbuch der Ingenieur-wissenschaften." Edited by Schäffer & Sonne. Vol. 2, Chap. 10. Leipzig, 1882. (Illustrates influence curves for moments and shears on simple and continuous girders.)
9. Winkler. "Über die Belastungs-Gleichwerthe der Brückenträger." 1884. (Use of influence lines in determining the equivalent uniform load for bridges.)
10. Winkler. "Aeussere Kräfte der Balkenträger." Wien, 1886. Third edition. (Second edition, 1872.) (Application to shears and moments in simple and continuous girders.)
11. Müller—Breslau. "Die neueren Methoden der Festigkeitslehre." Leipsig, 1886. (Some applications to continuous girders, and to frames with superfluous bars.)
12. Levy. "La Statique graphique et ses applications aux Constructions." Paris, 1886. second edition. (Numerous applications of influence lines are given in this extensive work, the second edition of which reached the author's hands since the completion of this paper. Part II treats of applications to simple and continuous girders, and in the preface further applications to arches in the parts to follow are promised.)

PLATE I
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Fig. 1.

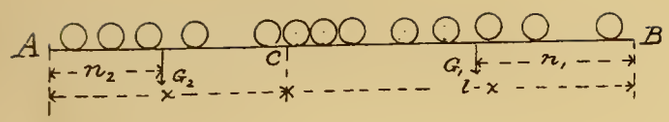



Fig. 2

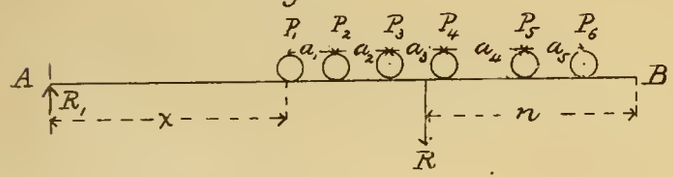



Fig. 3.

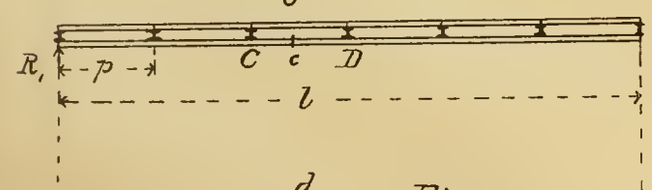



Fig. 4.

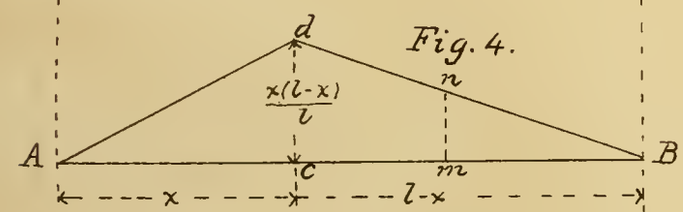



Fig. 5.

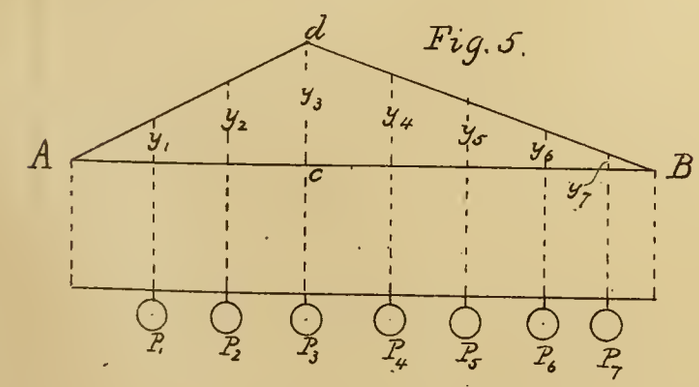



Fig. 6

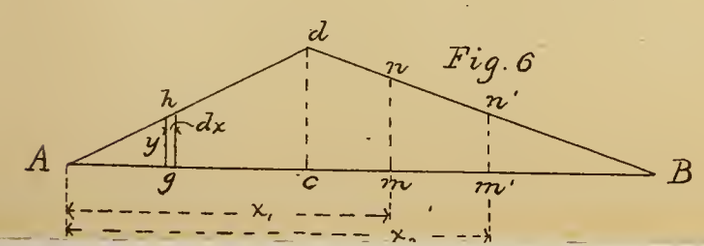



Fig. 7.

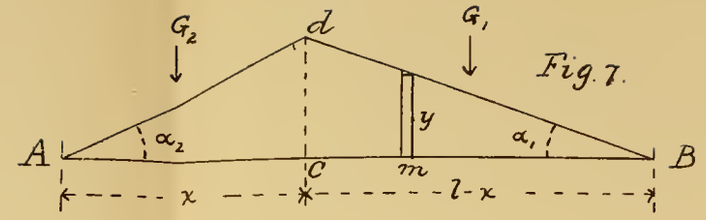



Fig. 8.

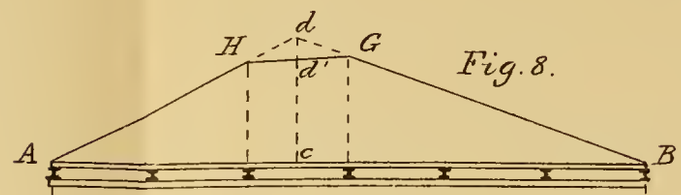
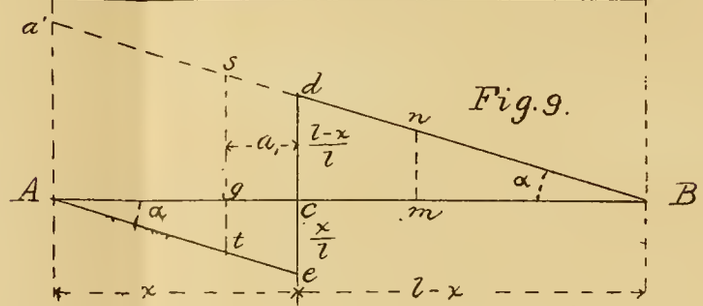



Fig. 9.



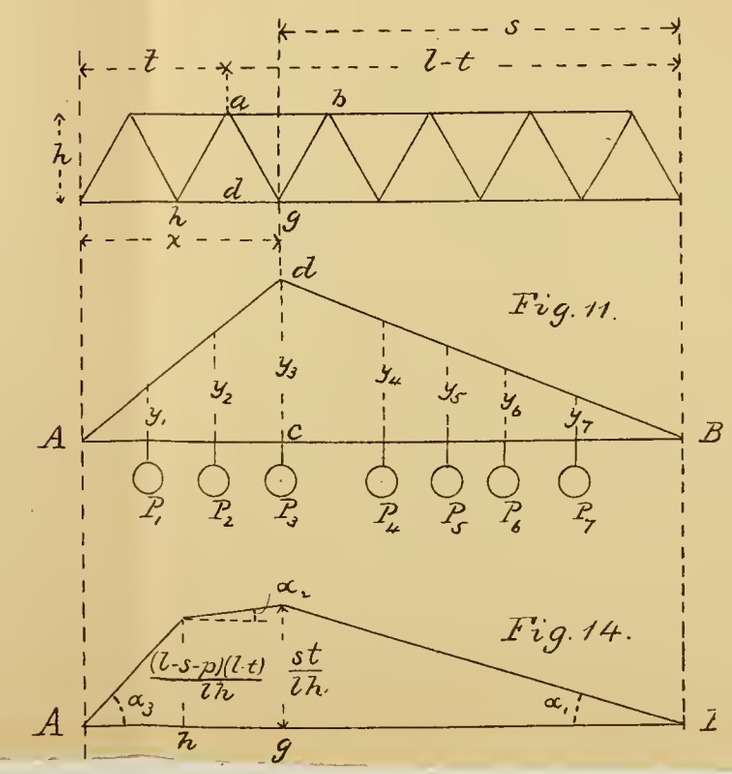
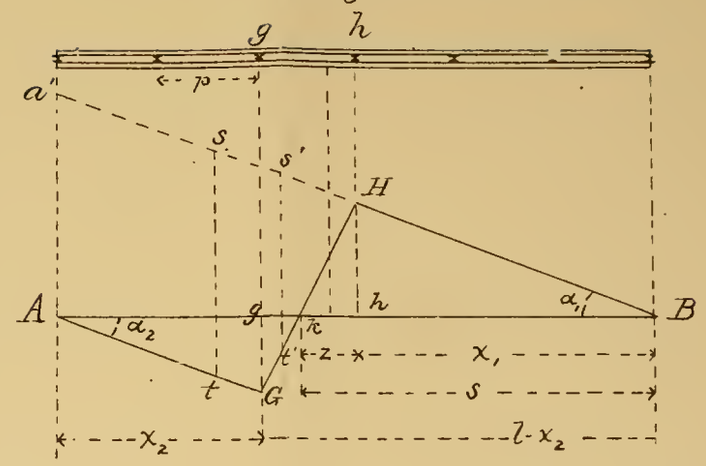




Fig. 10



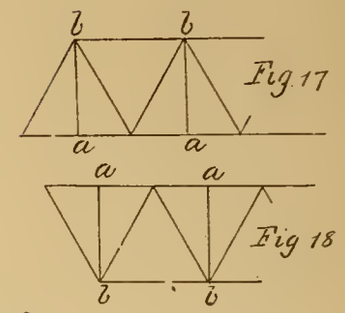
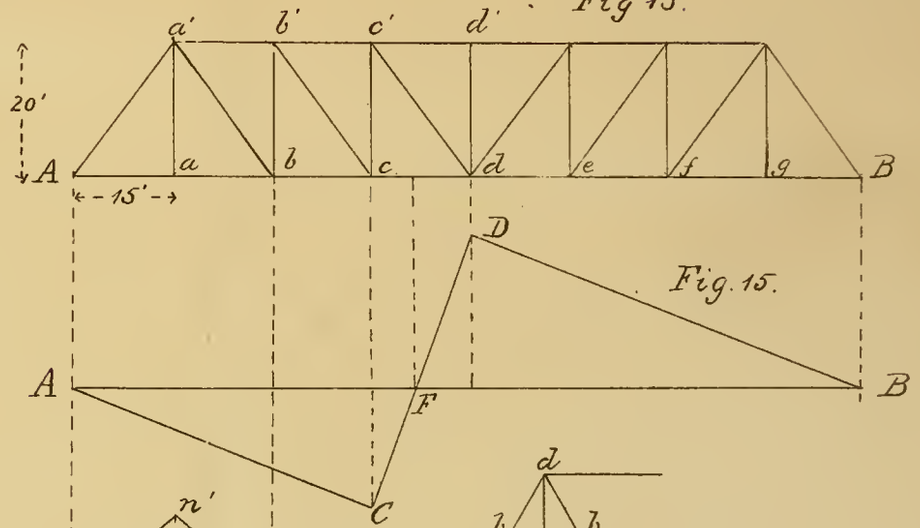




Fig 13.



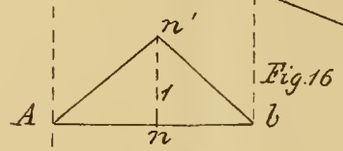




Fig. 16

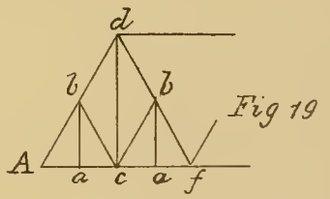



Fig 19

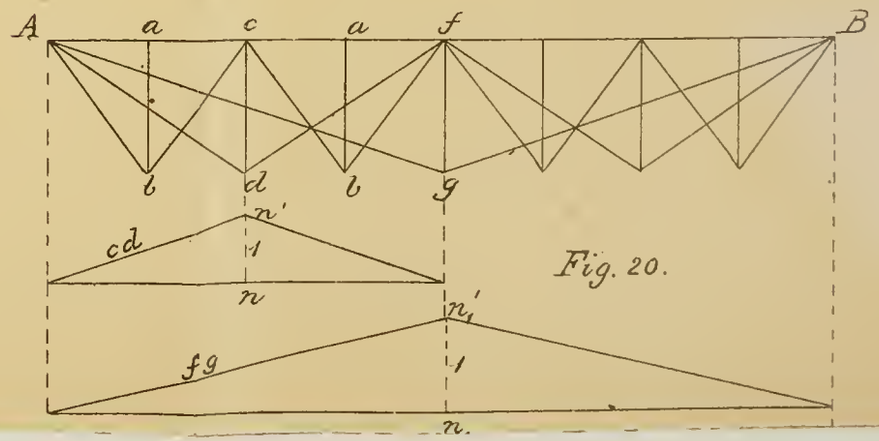



Fig. 20.

Fig 12.

| | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Moments | 0 | 45 | 126 | 261 | 450 | 1017 | 1327 | 1712 | 2102 | 2790 | 3480 | 3948 | 4470 | 5046 | 6516 | 7256 | 8114 | 8934 | 9450 | | |
| Total lds. | 6 | 18 | 30 | 42 | 54 | 62 | 70 | 78 | 86 | 92 | 104 | 116 | 128 | 140 | 148 | 156 | 164 | 172 | | | |
| Distances | | 7.5 | 12 | 16.5 | 21 | 31.5 | 36.5 | 42 | 47 | 55 | 62.5 | 67 | 71.5 | 76 | 86.5 | 91.5 | 97 | 102 | 105 | | |
| Loads | 6 | 12 | 12 | 12 | 12 | 8 | 8 | 8 | 8 | 6 | 12 | 12 | 12 | 12 | 8 | 8 | 8 | 8 | | 3000 lbs per ft. | |

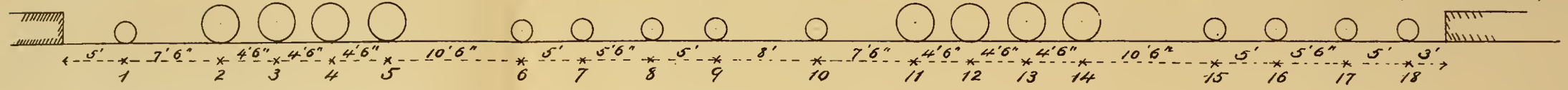



Fig. 21.

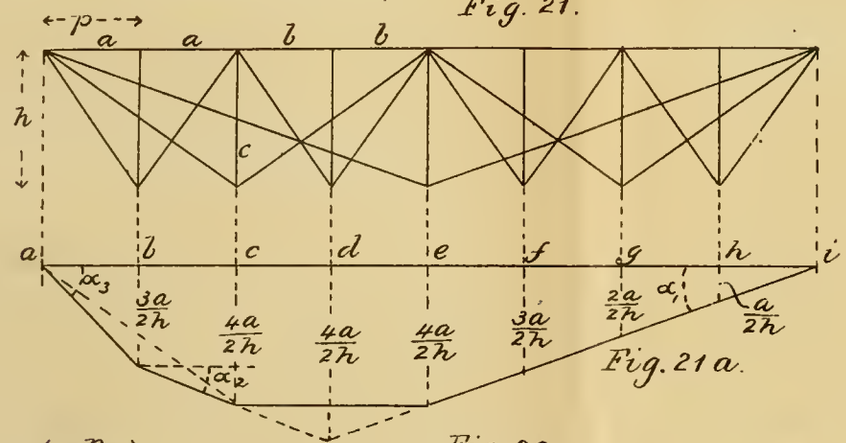



Fig. 21a.

Fig. 22.

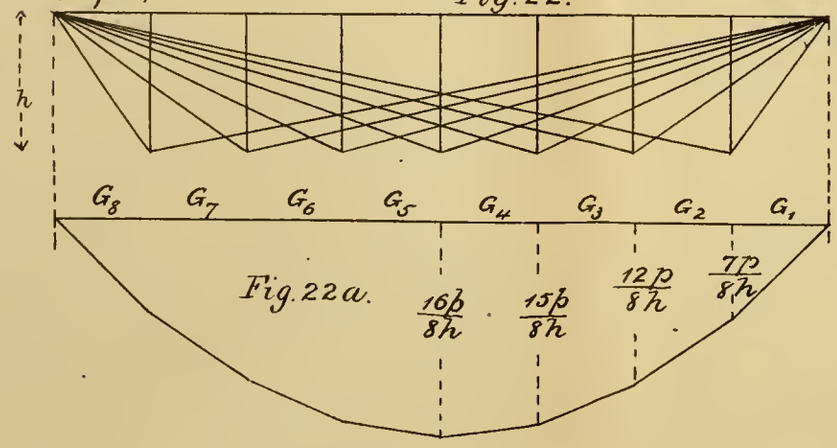
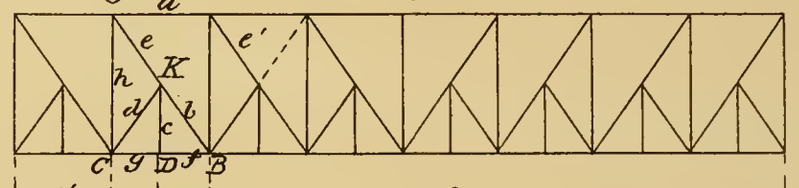



Fig. 22a.

Fig. 23.



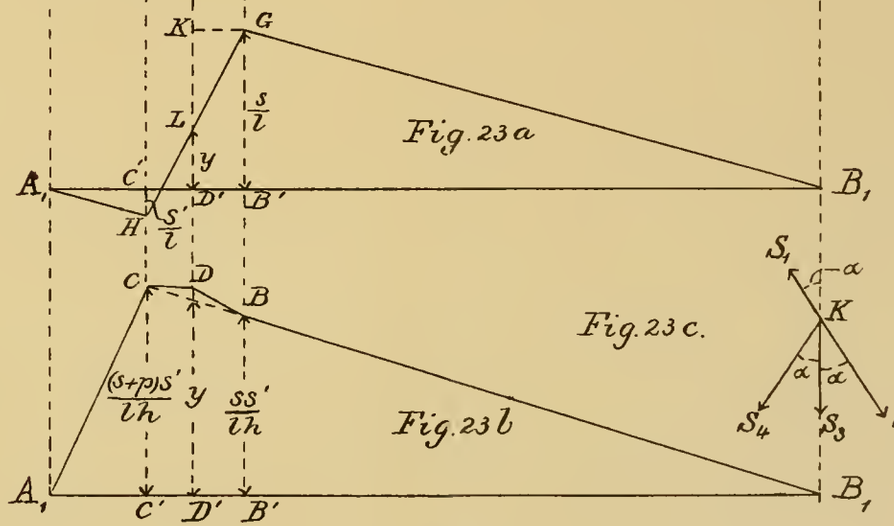




Fig. 23a

Fig. 23c.

Fig. 23b

Fig. 24.

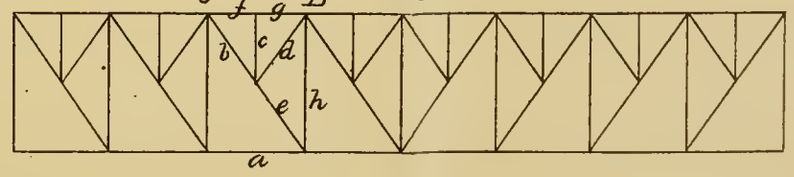


PLATE III
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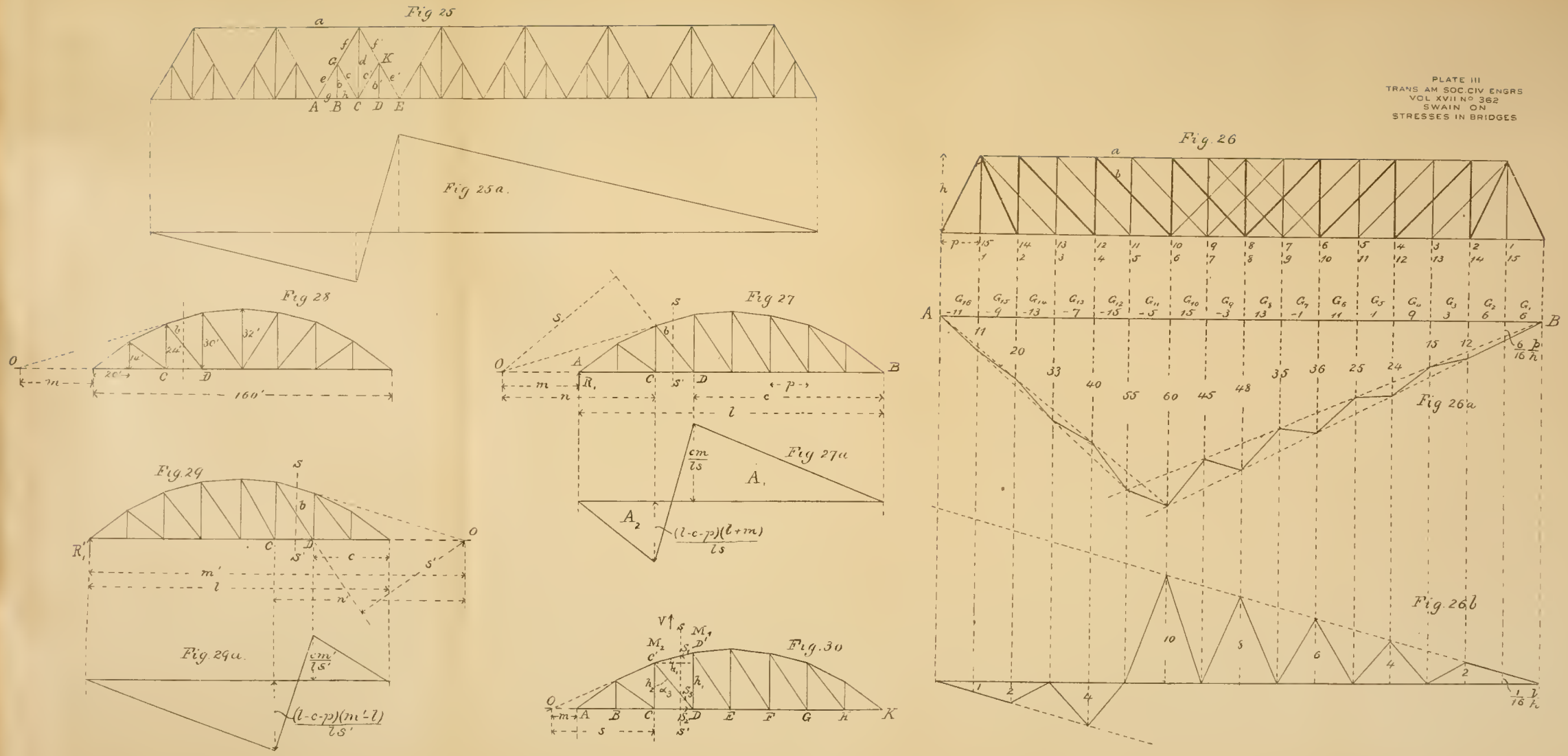

PLATE IV
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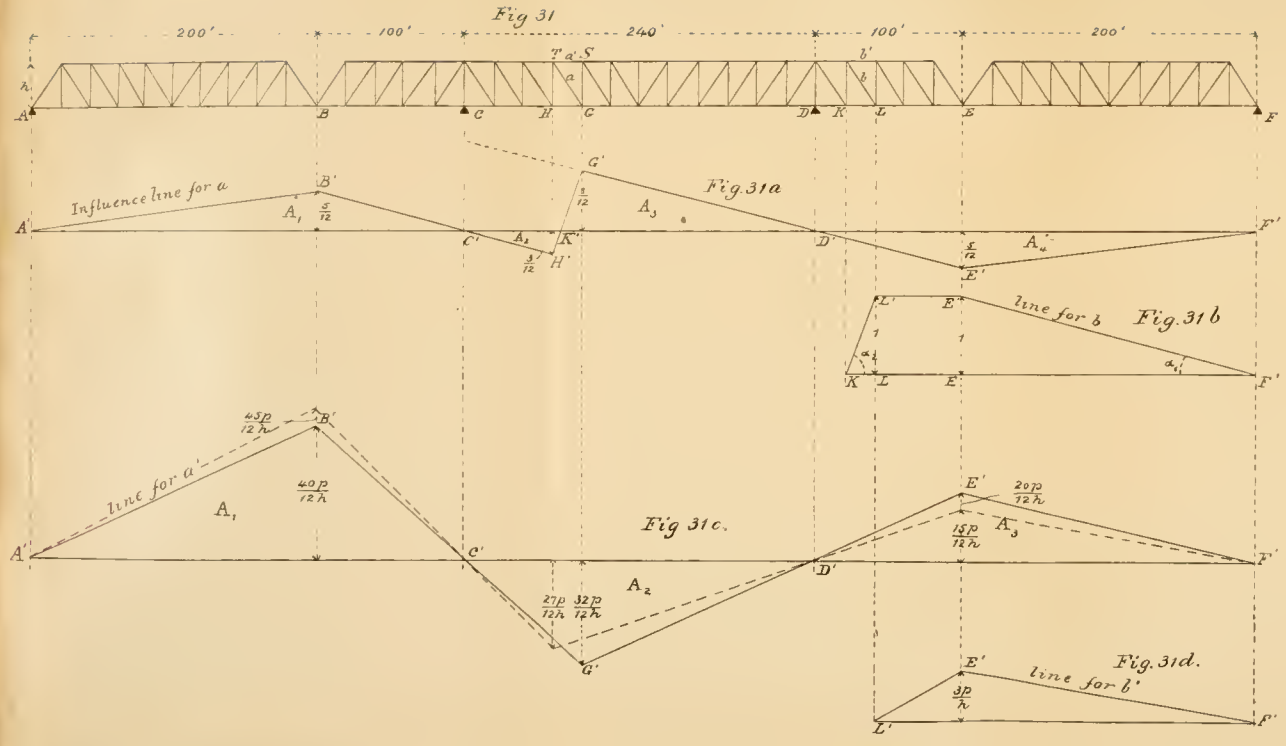


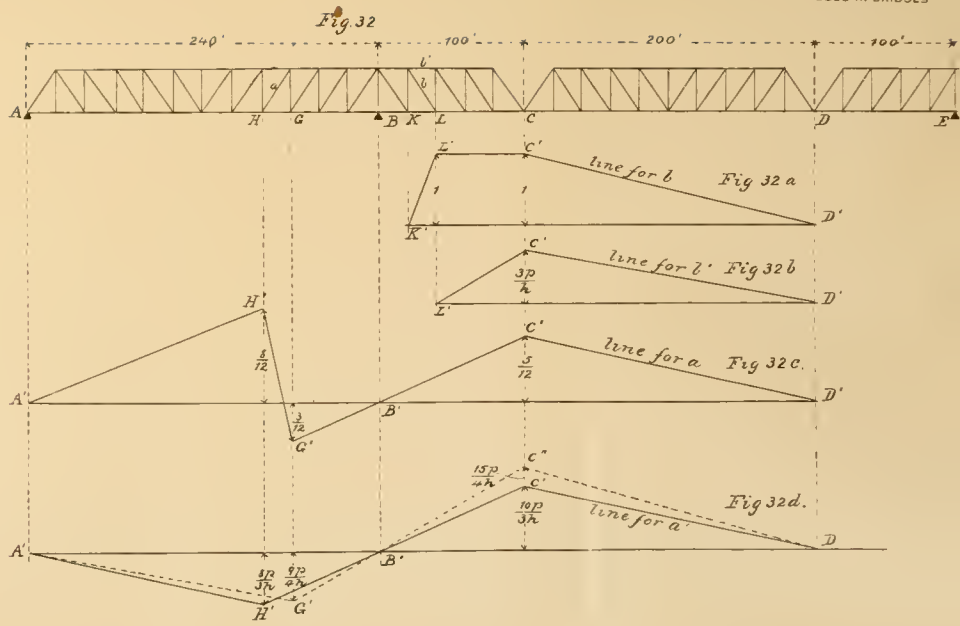

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced any of its publications.

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(Vol. XVII.—August, 1887.)

# COMPRESSIVE STRENGTH OF STEEL AND IRON.

BY CHARLES A. MARSHALL, M. Am. Soc. C. E.

PRESENTED FOR THE ANNUAL CONVENTION, JULY 1ST, 1887.*

The experiments which form the basis of this paper were undertaken with the object of discovering the relation which compressive strength of steel bears to tensile strength. The results appeared to warrant extending the investigation to include wrought-iron, and sufficient tests were made to exhibit the analogy between the two kinds of material.

No attempt has been made to test full sized members of structures, nor any shapes but solid rolled bars as they came from the rolls, except in a few cases where the bars were turned. These were chosen because they are the simplest possible shapes, and it was thought that by eliminating complexity of form and all treatment of the specimens, the properties of the material would be less liable to be obscured. They also possess the advantage of cheapness; are easily obtainable; and permit of a number of tests being made from the same rolled piece, which is not so practicable with larger shapes. Notwithstanding the limitation as to shape, the results are believed to be of value, because they clearly show the relation sought and indicate what appears to be a general law, which is at least clearly demonstrated by the tests so far as solid bars are concerned, namely, that the elastic limit of the material is the chief factor in determining the ultimate resistance of struts of ordinary length made out of

* This paper was presented at the Annual Convention of the Society, July, 1886, and afterwards withdrawn and rewritten by the author for the Convention of July, 1887.

wrought-iron or steel, excepting the very hardest kinds; and that the two quantities, elastic limit in compression and ultimate compressive strength, are identical within a considerable range of length-ratio of columns.

The writer wishes to state that he was first directed to this law by Mr. James E. Howard, well known as the engineer in charge of the testing machine at Watertown Arsenal. In reply to the query, "What, from his experience, he would say is the compressive strength of wrought-iron?" that gentleman replied that he thought it was about equal to the elastic limit of the material. This view was kept in mind and special attention paid to elastic limit in these tests, and no compression test is included without a corresponding tension test.

The following enumeration of Tables and Diagrams giving results of the tests will afford an idea of the ground covered and facilitate reference:

TABLE No. 1.—Seventy-four flat-ended compression tests of 28 different sizes of bars, .44 to 6.0 square inches section, all from same blow of Bessemer steel. Lengths principally 2, 12, 18, 24 and 30 times shortest diameter.

TABLE No. 2.—Average results of full size tensile tests of the 28 different bars used in tests of Table No. 1.

TABLE No. 3.—Twenty-one flat-ended compression tests of wrought-iron bars, 9 different sizes, made from scrap. Length 2, 12, 18, 24, 30 and 36 times shortest diameter.

TABLE No. 4.—Fourteen tensile tests of the 9 bars of Table No. 3, made on full size pieces cut from each bar.

TABLE No. 5.—Thirty-seven flat-ended compression tests of small steel bars from 18 different casts of steel, ranging from the softest open-hearth steel of 46 000 pounds tensile strength* to high carbon steel of 144 000 pounds tensile strength, also seven tests of two different qualities of iron. Lengths principally 3 times and 12 times the diameter.

TABLE No. 6.—Tensile tests of each bar or set of bars used in tests Table No. 5. Those bars bearing laboratory numbers from 329 to 559 were rolled in sets of four from each heat and cut into test pieces, which were afterwards numbered indiscriminately. Several of each set had been broken, showing very uniform results.

TABLE No. 7.—Compression tests of bars 12 inches long by 1 inch square, cut from each one of 9 rods rolled from the same blow of Cambria 70 000-pounds Bessemer steel; also similar tests from each of 20 bars from a uniform lot of iron purchased from Union Iron Mills of Pittsburg.

TABLE No. 8.—Hinge-ended compression tests of 34 specimens 1-inch square as from rolls, lengths varying from 16 to 60 inches, cut from the same steel rods described under Table No. 7. These will be referred to as "Series W."

TABLE No. 9.—"Series K," in all respects similar to Series W, specimens being cut from the iron rods described under Table No. 7.

---

* All statements of stresses and strengths in this paper are in pounds per square inch, the actual loads being omitted for the sake of brevity.

TABLE No. 10.—Detail of tensile test of 1-inch square steel from bar W 5, Series W.

TABLE No. 11.—Detail of compression test of 1-inch square steel 4.02 inches long from bar W 5, Series W.

TABLE No. 12.—Detail of tensile test of 1-inch square iron from bar K 11, and summary of tensile test of K 5, Series K.

TABLE No. 13.—Detail of compression test of 1-inch square iron, 3.984 inches long, from bar K 11, Series K.

TABLE No. 14.—Detail of compression test of laboratory No. 2 336a, 1-inch round steel 12 inches long. (Compare Table No. 5.)

The foregoing will be found at end of the paper.

TABLES Nos. 15, 16 AND 17.—Moduli of steel and iron bars in tension and compression, described where introduced.

Plates V, VI, VII, VIII and IX give test diagrams for several of the different classes of material experimented with, the principal results of the tests being also recorded in Tables Nos. 5 and 6. In each case the tensile test is plotted as ordinarily, and also plotted so as to show stress per square inch upon the reduced diameter, abscissas representing percentage of stretch and ordinates the stresses. The compression test of a round specimen 3 diameters long is similarly plotted to exhibit stress upon enlarged diameters, as well as referred to original diameter. The early part of the tensile test, including passage of the elastic limit, is shown also with the horizontal scale magnified ten times, and with it is given the compression test of a specimen usually 12 diameters long.

Plate X is a graphical exhibit of the maximum resistances of bars covered by Table No. 8, Series W.

Plate XI is a graphical exhibit of the maximum resistances of bars covered by Table No. 9, Series K; also graphical exhibit of a series of tests on 3-inch square iron bars made at Watertown Arsenal, a summary of which can be found on page 118 of the Report of Tests at Watertown Arsenal for 1883.

Plate XII shows the form of rocker-bearing used in the hinge-end tests. Its peculiarity is that the axis of the pin is in the plane of end bearing, square ended-specimens being used. Thus, while saving the expense of boring the specimens, all doubt as to effective length of specimen is eliminated, and the pin friction has its proper effect the same as if specimens were bored.

The pins are of hard steel, well fitted and lubricated with a mixture of plumbago, tallow and oil; hence, although their diameter is large as compared with the specimens used, pin friction is very small.

All the tests, except those from the Watertown Report referred to above, were made in the testing laboratory of the Cambria Iron Company, Johnstown, Pa., by whose kind permission I am enabled to lay the results before this Society.

A vertical Emery Testing Machine of 300 000 pounds capacity was used for all except a very few of the smaller tensile specimens, which were tested on a Gill machine of 100 000 pounds capacity.

All specimens whose modulus of elasticity is recorded in the tables had their change of length up to and for a short distance beyond the elastic limit, measured with an electric double micrometer designed by the writer, which reads to the $\frac{1}{10\,000}$ of an inch, though it is not claimed

to be thoroughly reliable closer than .0001 inch. The mode of testing is fairly exhibited in Tables Nos. 10 to 14.

The diagrams were plotted accurately on engraved cross-section paper. They were traced and the plates are photo-lithographs of the tracings.

The short compression specimens were removed and measured with a measuring machine for shortening after each load had been applied one minute or longer, usually a minute, and were measured a number of times during later part of test to determine enlarged area. The diagrams "per square inch actual" were thus not so completely determined as the others, but are however accurate enough to exhibit fairly the comparative behavior under tension and compression. After passing the elastic limit the molecular stress is seen to be greater for a given change of length under compression than under tension, until near the end of tensile test the ultimate tensile molecular stress is seen to be greater than could be obtained with specimens 3 diameters long, except in case of the spring steel, where the two are nearly equal. The closest agreement is in the case of the ductile stay-bolt iron, and the widest divergence in case of the hard iron made from scrap.

Before referring more in detail to the tests, it will be well to define elastic limit as noted in the tables and as used in this paper. Wherever there are no qualifying terms, the limit referred to is what is sometimes called principal elastic limit, sometimes primitive elastic limit.

"Elastic limit by beam" is the same thing when properly taken on soft material.

"Elastic limit by micrometer," in the few cases of hard steel where it is noted, is that point where the change in increment of length due to a certain increment of load is most marked, that being the phenomenon which bears closest resemblance to the principal elastic limit of softer material, the difference being a difference of degree.

In connection with this part of the subject, attention is directed to two phenomena exhibited clearly in all the diagrams of Plates V to IX excepting that of the spring steel. *First.*—When a load equal to the elastic limit of any of these materials is imposed and allowed to remain for some time, it causes permanent change of length, amounting, it is believed, to much more than has been generally understood, and which is apparently a very definite quantity for each case. *Second.*—This change of length having begun under a load equal to elastic limit, will continue under a less load.

The plotted tests in tension were all so made as to develop the amount of this stretch; and it is demonstrated in the cases of the 100 000-pound steel, the 67 000-pound steel, and the hard iron made from scrap, that the total is obtained equally as well, though requiring greater length of time, with a load 1 000 or 500 pounds less than primitive elastic limit as with a load equal to that limit. This becomes

clear from the diagrams, when it is understood that the loads just above the limit were each allowed a minute or more in which to develop stretch.

Time is noted on some of the diagrams sufficiently to give a general idea of the variations in speed of stretching under different loads and at different periods. It is seen that for the same duration of application the less loads cause less change of length. In all the tension and some of the compression diagrams will be seen points where the load was partly taken off and then raised by increments in order to exhibit the second of the phenomena referred to.

The amount of stretch at elastic limit as taken from the tension diagrams:

| | | |
|---|---|---|
| For the 100 000-pound steel | 0.3 | per cent. |
| " 80 000 " | 1.4 | " |
| " 67 000 " | 1.85 | " |
| For hard iron | 1.3 | " |
| " soft iron | 2.3 | " |

It is to be regretted that the same mode of testing was not followed with the shortest compression specimens, but in those which are plotted each load was allowed to act for only about one minute, hence the exact amount of change of length at limit is not clearly shown. It apparently varies from near equality with that under tension to about half as much. The soft iron shows near equality, the hard iron considerably less; the steels near equality, excepting, perhaps, the 80 000-pound steel. The detail test reports, Tables Nos. 10 to 13, furnish the following accurate comparison:

| | | |
|---|---|---|
| In tension, steel W has | 2.11 | per cent. |
| " compression " " | 2.05 | " |
| " tension, iron K " | 1.38 | " |
| " compression, " " | 0.57 | " |

It is noticeable that the specimens under compression reach their maximum change of length under constant load in less time than those under tension. This appears to be due partially to more rapid action, but is more attributable to difference in length of specimen.

These phenomena may appear trivial to some, but the writer considers that they furnish the key to his subject, and hence attention is further directed to the behavior of the specimen at elastic limit.

Let us follow the course of an experiment for determining elastic limit without the use of measuring instruments. With steel bars having roll scale on, this can usually be done with certainty and dispatch, by setting the speed of straining very small and watching the gradual and continual rise of load till suddenly the load either ceases to rise or else drops materially. Either the first phenomenon or the second is taking place and, as all familiar with testing would recognize, the elastic limit is reached. It is "elastic limit by beam," and this mode of test is prob-

ably more used than any other. Watch should always be kept for the appearance of "scaling," that is the breaks in continuity of the roll scale, which it appears is able to stretch without breaking (in case of steel) up to elastic limit of the steel, but is far too brittle to maintain its integrity under anything like one-half per cent. of stretch. The roll scale on iron seems to be more brittle than is usual on steel, and sometimes begins to drop off before elastic limit is reached. When considerable scaling near ends of specimen and drop of beam first occur simultaneously, the indication of elastic limit is absolute. Certain important conditions, however, are necessary in using this mode of determination.*

For an example of how minute a permanent set is sufficient with the apparatus used to indicate elastic limit by this method see Table No. 11, where a correct indication was had, and the specimen being removed immediately, showed .0035 inch total shortening.

It is of importance to note that this change of length of which we have been speaking, is invariably local and progessive in its action. Thus the scaling can be watched as it proceeds from the ends of a tension or compression specimen towards the center, and the writer has often demonstrated, by means of micrometer measurements, that the central portion of the piece is undergoing no permanent change of length so long as the roll scale remains unbroken between measuring points, and also that as soon as the progressive action has proceeded till the lines of rupture of scale meet at the middle of length, the whole attainable stretch has been had, except an inappreciable fraction.† The load usually fluctuates while this is going on.

If now one tries to imagine a free strut of, say, 20 diameters long, of whatever shape undergoing this change of length, bearing in mind its local progressive character, it requires no great effort of reason to conclude that if the strut at end of the shortening (when it is ready to show increased resistance) is straight, such would be a purely accidental result.

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* Thus if the roll scale is red in color from whatever cause, it will afford a poor indication of elastic limit; such scale or rust possesses very little coherency in itself, and is more strongly attached to the metal. Also a loose rising scale is not permissible. Reheating or tempering spoils the scale for this purpose.

The indication is not accurate with very hard materials; it may be used as a guide and comparative test of steels, but does not furnish a proper quantity to base deductions or calculations upon as regards the cons'ructive value of the material. (See Tables Nos. 5 and 6 for examples of spring steel.) Some irons, such as are granular or brittle, may easily cause erroneous tests by this method.

The specimen must not be crooked, and the grips must be in good condition. This, however, is of less importance when the roll scale is of the best character, since by waiting for appearance of the scaling on body of specimen, and noting the load at that time, the proper determination can be nade.

The speed of straining must be well known by experience, and must never be too great.

The writer uses an instrument to show change of length whenever there is danger that any of the conditions are unsuitable.

† A curious fact is that these lines develop themselves at an angle of 45 degrees in tension and 90 degrees to axis in compression usually.

In a very few of the steel flat-ended tests recorded in the tables, where the length did not exceed 12 diameters, such a condition existed; these specimens usually assumed later an S-curve and gave a little higher maximum load than those which behaved in the usual way, *i.e.*, which bent at some point soon after scaling commenced at the ends, and proceeded to scale on the interior of curve each way toward ends as well as from the ends on the opposite side, producing the well known reversed curve tangent to axis at the ends common to flat and fixed-ended struts, and which is called in the tables "orthodox." The iron specimens in Table No. 7, Series K, afford several instances of a fair degree of straightness being maintained. The other irons tested afford comparatively few, but on the whole iron bars seem more likely to behave in that manner than steel.*

In Tables Nos. 1, 3, 5 and 6, are columns whose heading requires some explanation. That called "Load per square inch at failure," is neither more nor less than elastic limit "by beam," as above described. But in view of the subsequent falling of the load and considerable bending that usually takes place, and of the fact that the same load is the maximum in every case in those tables of a bar longer than 12 diameters, it is described as "failure." The phenomenon certainly impresses one as failure when it is seen. It is not total failure, nor in all cases final failure, but it is failure for the time being to sustain the maximum hitherto imposed load. By continuing the test in those cases where the bar receives an accession of stiffness caused by the most highly distorted portions having completed the change of state at primitive elastic limit, and taken on a new higher limit, sometimes, and more often in case of iron than steel, another maximum load is obtained, sometimes, though not always, exceeding the load at first failure. This is noted as "second maximum." In some cases perhaps a mere fluctuation of elastic limit was mistaken for a true second maximum. The second maximum is likely to be higher in bars which remain straight or take an S-curve during passage of elastic limit, as described in the preceding paragraph.

Turn now to Tables Nos. 1 and 2. These are held to furnish clear and positive proof that crippling strength does not depend upon nor bear any definite ratio to ultimate tensile strength, while the close agreement of the quantities in columns 7, 8, 9 and 10 establish the first part of the general law stated in the second paragraph, or to state it again in slightly

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*It sounds paradoxical, but the explanation probably is because of the greater homogeneity of the steel. Thus the boundary surface between the crippled and uncrippled portions of metal in case of steel will be regular and comparatively smooth, and in case of the fibrous and less homogeneous iron may be irregular and covered with indentations, or points of uncrippled metal projecting into the crippled or flowed metal. It is not difficult to see how this may cause the iron to keep in line better. The layer or extremely short prism of iron or steel undergoing change of state at any instant may be regarded as in unstable equilibrium.

different language, the crippling strength of steel in the form of moderately long struts is approximately equal to the elastic limit of the material. Column 7 was derived from specimens only two diameters long. The tendency is with very short specimens, either in tension or compression, to get too high values for elastic limit. Nevertheless the greatest excess of any qauntity in that column over the corresponding tensile elastic limit is 3 423 pounds and the average excess is 1 370 pounds. The quantities in column 9 for specimens 12 diameters long are believed to be more closely the true compressive elastic limit. The greatest differences between any corresponding quantities, column 9 minus column 8, are + 2 620 and — 1 632, with an average difference of about + 480.

Inspection of column 10 in comparison with column 9 discloses that rarely does the bar exhibit any higher resistance than the elastic limit when 12 diameters long or over. By referring to Column 6 it will be seen that such bars as failed to show crippling strength as great as elastic limit in tension, generally had some defect; or, as noted in one instance, there was failure to stress squarely.

The material for these tests all possessed the same constitution, being from one blow of steel, and is very uniform, as shown by the small variation in ultimate strength, less than 6 000 pounds, notwithstanding the wide range of thickness and sectional area. The tables bring out the fact that elastic limit varies more with size of bar than does any other property of the metal, and there are but two exceptions to the rule that the thicker the bar the lower the elastic limit and ultimate strength.

Table No. 4 shows that the same law applies to iron, though the exceptions are more numerous and the material evidently much less uniform than the steel. Elastic limit varies 11 260 pounds and ultimate strength 9 280 pounds (excluding specimen 2 193, whose low strength is evidently caused by a sliver on the corner of bar). There are some irregularities in each series to be accounted for by irregularities in the heating and rolling. Neither is offered as exhibiting the proper numerical relations of strength, etc., of bars of the same material differing in size.

In Table No. 3 each compression test in column 9 of a piece 12 diameters long is in excess of the corresponding tensile elastic limit, the maximum excess being 1 905 and the average 880 pounds. Out of 13 cases 12 diameters long all but two have a second maximum higher than elastic limit in compression.

Tables Nos. 5 and 6 need little explanation. They serve to extend the application of our law from the softest material called steel up to steel of 100 000 pounds strength, thus covering the whole range of structural steels. Spring steel of 144 000 pounds strength is an exception to the law. The consideration that change of state takes place very gradually and at no one definite load, causing a gradual rounding

of the diagram without any such horizontal portion as shown by the other materials (notwithstanding efforts were made to develop it) shows this to be the exception that proves the rule.

Table No. 7 calls for no further explanation, save that one case of a hinge-ended bar, which bent along axis, is included as flat-ended. The average excess of maximum resistance over compressive elastic limit for flat-ended steel bars is 757 pounds; hinge-ended steel bars, 395 pounds; flat-ended iron bars, 3 171 pounds; and hinge-ended iron bars, 1 807 pounds.

Before discussing the series of bars of varying length whose results are plotted, the modulus of elasticity claims attention, This property of material has been the subject of much loose thought, and widely discordant data have been made public. Many of the widely varying figures, quoted as modulus of elasticity, have been deduced from experiments in bending stress by the common theory of beams, and considering that this theory yields values for ultimate tensile or compressive stress which are in error sometimes to the extent of 50 per cent. or more, we are justified in throwing aside this as no method of determining Young's modulus.

Engineers are to-day, however, in possession of enough data derived from direct tensile experiments to give fairly correct averages for iron and steel. Still in one point many deductions are incorrect, because the total change of length upon first application of loads has been used in the calculations, instead of the change of length recovered upon release of load. The quantity determined by the first of these two modes of calculation will be called modulus of extension or of compression, as the case may be. That by the second mode is the modulus of elasticity ($E$) in tension or in compression. The two differ more widely in long and large pieces than in light sections, and more in iron than steel, so far as the writer's observations have extended. The cause of the difference is doubtless the presence of internal strains, which are more or less relieved by introduction of permanent sets upon even a single application of stress, and which are greater in large and complicated sections than light ones, and to all appearances greater in iron than steel, comparing the same size of pieces. The difference under tension on small bars is on the average very small.

In Tables Nos. 15 and 16 following, have been collated for comparison all the available determinations elsewhere appearing in this paper. The averages in tension bear out the last statement. The range of load upon which calculations were based varied somewhat, but may be roughly stated as from 5 000 pounds per square inch to three-fourths of the elastic limit. Above that load the modulus of extension will often decrease in value, though many specimens hold to one modulus, very close up to elastic limit. As regards the true modulus $E$, it is sensibly constant up to the elastic limit, and its variations for different kinds of steel are insignificant.

## TABLE No. 15.

Comparison of Moduli for Tension and Compression Steel Bars.
First: Ultimate strength 100 000 or under.

| Laboratory No. | Size bar. Inch. | E. Tension in 1 000 lbs. | Modulus of Extension in 1 000 lbs. | Laboratory No. | E. Compression in 1 000 lbs. | Modulus of Compression in 1 000 lbs. | Remarks. |
|---|---|---|---|---|---|---|---|
| 329 | 1 rd. | 30.420 | 30.190 | 331 | 29.740 | 29.450 | |
| 346 | $\frac{8}{10}$ sq. | 29.850 | 29.850 | 348 | 29.010 | 28.070 | |
| 484 | 1 rd | 29.500 | 29.280 | 476 | 29.420 | 28.780 | |
| | 1 rd. | ..... | ..... | 479 | 29.200 | 28.950 | |
| 455 | $\frac{3}{4}$ rd. | 29.150 | 29.830 | 457 | 29.420 | 28.580 | |
| 464 | $\frac{3}{4}$ rd. | 29.800 | 29.670 | | | | |
| 497 | $\frac{8}{10}$ sq. | 29.640 | 29.420 | 503 | 28.670 | 28.380 | |
| 2336 | 1 rd. | 29.630 | 29.550 | 2336a | 28.830 | 28.680 | |
| 2352 | 1 rd | 29.960 | 29.240 | 2352a | 30.490 | 30.070 | |
| 2345 | 1 rd. | 30.420 | 29.400 | 2345a | 29.790 | 28.980 | |
| 2337 | 1 rd. | 30.370 | 30.000 | 2337a | 29.810 | 29.260 | |
| W 5 | 1 sq. | 30.420 | 29.630 | | | | |
| 1667 | $\frac{3}{4}$ rd. | ..... | ..... | 1667a | 27.880 | 27.590 | |
| | 1 rd. | ..... | ..... | 1669a | 27.590 | 28.780 | |
| | $\frac{3}{4}$ sq. | ..... | ..... | 1595a | ..... | 27.740 | |
| | 1 sq. | ..... | ..... | 1587 | 28.950 | 28.080 | |
| | $1\frac{1}{4}$ sq. | ..... | ..... | 1583 | 28.920 | 28.780 | |
| 1701 | $3 \times \frac{3}{16}$ | ..... | 29.350 | ..... | ..... | ..... | Strip from bar. |
| 1702 | $3 \times \frac{3}{16}$ | ..... | 29.640 | ..... | ..... | ..... | " |
| 1721 | $3 \times 1\frac{1}{2}$ | ..... | 29.410 | ..... | ..... | ..... | " |
| 1722 | $3 \times 1\frac{1}{2}$ | ..... | 29.860 | ..... | ..... | ..... | " |
| | | 29.928 | 29.621 | ..... | 29.123 | 28.678 | Average. |

Second: High Carbon Spring Steel. Ultimate strength 144 000.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 512 | 1 sq. | 29.960 | 29.850 | | | | |
| 548 | 1 rd. | 29.760 | 29.480 | 559 | 29.330 | 28.880 | |
| 549 | 1 rd. | 29.580 | 29.390 | 547 | 29.200 | 28.880 | |
| 584 | 1 sq. | 29.850 | 30.530 | | | | |
| 576 | $\frac{8}{10}$ sq. | 29.420 | 28.880 | 573 | 29.220 | 29.090 | |
| 574 | $\frac{8}{10}$ sq. | 29.200 | 29.200 | 579 | 29.350 | 29.090 | |
| | | 29.462 | 29 555 | .... | 29.275 | 28.985 | Average. |

## TABLE No. 16.

Comparison of Moduli for Tension and Compression—Iron Bars.

| Laboratory No. | Size bar. Inch. | E. Tension in 1 000 lbs. | Modulus of Extension in 1 000 lbs. | Laboratory No. | E. Compression in 1 000 lbs. | Modulus of compression in 1 000 lbs. | Remarks. |
|---|---|---|---|---|---|---|---|
| 2157 | rd. | 27.500 | 26.800 | 2158 | 26.160 | 25.840 | |
| 2160 | rd. | 27.410 | 26.980 | 2159 | 26.240 | 25.920 | |
| 2161 | 1 rd. | 26.700 | ..... | 2162 | 26.440 | 25.670 | |
| 2165 | 1 rd. | 27.540 | 27.540 | 2163 | 26.350 | 26.020 | |
| 2195 | sq. | ..... | 28.990 | 2196 | ..... | 27.420 | |
| 2198 | sq. | 29.180 | 27.790 | 2197 | 27.790 | 25.650 | |
| 2167 | 1 sq. | 27.900 | 27 800 | 2168 | 27.300 | 26.490 | |
| 2171 | 1 sq. | 28.290 | 28.290 | 2169 | 27.990 | 27.100 | |
| 2306 | 1 rd. | 28.570 | 27.590 | 2307 | 28.570 | 27.250 | |
| 2308 | 1 rd. | 28.480 | 28.290 | 2309 | 28.570 | 27.430 | |
| 2310 | 1 rd. | 28.480 | 26.580 | 2311 | 28.180 | 26.500 | |
| 2390 | rd. | 30.190 | 30.190 | 2391a | 29.910 | 29.520 | |
| | rd. | ..... | ..... | 2391b | 30.120 | 29.710 | |
| K 11 | 1 sq. | 27.910 | 28.070 | | | | |
| | | 28.179 | 27.992 | ..... | 27.802 | 26.963 | Average. |

The following data from a large number of consecutive determinations of modulus of extension of standard ¾-inch round test bars, one from each heat of steel, made in 1885, show the narrow range of averages for the different kinds of steel.

TABLE No. 17.

MODULUS OF EXTENSION OF STEEL BARS.—CAMBRIA IRON COMPANY—TESTING LABORATORY, JOHNSTOWN, PA., 1885.

| Number of heats. | Average carbon. | Modulus of extension.—Pounds per square inch. | | | Kind of steel. |
|---|---|---|---|---|---|
| | | Lowest value. | Highest value. | Average value. | |
| 33 | .09 | 28 750 000 | 31 540 000 | 29 924 000 | Bessemer. |
| 89 | .34 | 28 140 000 | 30 910 000 | 29 672 000 | " |
| 8 | .11 | 29 210 000 | 30 670 000 | 30 020 000 | Open Hearth. |
| 107 | .27 | 28 310 000 | 31 180 000 | 29 996 000 | " |
| 25 | .72 | 28 680 000 | 30 860 000 | 29 919 000 | " |

Table No. 17, besides furnishing numerical values very close to the real modulus E, shows that Bessemer steel has lower average modulus of elasticity than good open-hearth steel. It shows also the general law that, other things being equal, the greater the content of carbon in the steel the lower is the modulus of elasticity.

Some further data extracted from the laboratory records, and relating to iron, may also be quoted:

Thirty tests on specimens cut from iron angles and channels of good quality ranged in modulus of extension from 26 740 000 to 30 510 000, with an average of 28 753 000.

Eleven tests of eye-bar iron of good quality ranged in modulus of extension from 27 510 000 to 30 370 000, with an average of 28 797 000.

Six tests of ⅜-inch plate-iron of not very good quality, ranged in modulus of extension from 25 880 000 to 27 000 000, with an average of 26 573 000.

The above determinations, including Table No. 17, were all made on Gill knife-edge testing machine with the same micrometer as used in the tests specially made for this investigation.

We come now to Tables Nos. 8 and 9 and Plates Nos. X and XI. Series W and series K are accurately comparable; size and condition of specimens, apparatus and mode of testing being the same in both. They differ from each other only in one being made of 70 000-pound steel and the other of bridge iron, and in the accidental crooks and bends incidental to the bars not having been cold straightened, the deviations due to which at the middle of length are recorded in the column "Condition of specimen." The initial deviation in direction 90 degrees from axis of pins is also written by the side of the plotted test result in the diagrams. On the diagrams are shown the range of elastic limit in compression, and the curve of resistances according to Euler's formula, modulus of compression being adopted from Tables Nos. 15 and 16, in round numbers 28 500 000 for steel and 27 000 000 for the iron bars. It is hoped that the tables furnish a tolerably clear condensed history of each test, enabling one to form an idea of the influence of pin friction and showing how various was the behavior, especially of the longer bars.*

These series, in connection with what has gone before, establish the second part of the law stated in our second paragraph, namely, that elastic limit in compression and ultimate compressive strength are identical within a considerable range of length-ratio of columns. Note that there is but one test in each series which falls below the horizontal belt of range of elastic limit, and that one is of the crookedest, and lies close to Euler's curve in each case.

As regards those results which fall to the right of Euler's curve, they are seen to scatter so widely that, their number being small, they cannot be considered as yielding much more than a suggestion as to how the subject should be studied. The horizontal belt above alluded to is marked by cross-lining, and the belt is extended to the right of the curve by making it to include all specimens whose initial deviation was

*A complete copy of the test records of those bars whose deflections were observed throughout is presented herewith. (See appendix to this paper.) No measurements of shortening were made except at end of test.

.02 inch or under, because bars with so much deviation very generally did as well as perfectly straight ones; bars with greater deviation sometimes do as well, but many drop almost to Euler's curve. The case looks hopeful that with a somewhat extended series of tests varied with reference to crookedness and other purposely introduced defects, such as lack of squareness of end-bearings, eccentricity of end-pressures, etc., quite definite results could be established.

From the closeness and approximate parallelism of lower boundary of the shaded belt to Euler's curve above $\frac{l}{r} = 165$ for the steel and $\frac{l}{r} = 140$ for the iron, it looks as if hereabout are the limits above which Euler's formula should be used pure and simple for hinged ends.

The pins used in these experiments (see Plate XII and description page 55) it is clear exerted very small end-fixing moment upon the bars; no reverse curvature was noticeable upon any above twelve diameters in length $\left(\frac{l}{r} = 42\right)$, so the results probably are on the safe side, supposing the results to be applied to pin-ended columns.

On Plate XI are plotted also the Watertown tests on 3-inch square bars. The range of elastic limit is obtained (probably imperfectly) from a set of three tests recorded in "Report of Tests," Watertown Arsenal, 1882, pages 206–208, stated on page 54 of 1883 Report to be of the same iron. The range of compressive elastic limits is inferred from the tests by the principles set forth in this paper. The main series had 1½-inch pins fitting half holes in ends of bars. Four tests flat-ended, and eight tests with other sizes of pins are also plotted. The enviable agreement of the pairs of tests is attributable in part to the bars having been straightened in a screw press as stated in the Report (see above), it being at the same time evidence of the extreme accuracy of the testing work, while the maintenance of a high strength with long specimens was doubtless aided by the comparatively great pin friction.

Probably the most important fact shown by these three sets of experiments is that the strength of the columns while equal to the elastic limit for a considerable range of length-ratio under that at which Euler's curve crosses the line of elastic limit falls off immediately on crossing the curve. Therefore, for pin-ended columns the length-ratio at which elastic limit ceases to be the measure of the column strength varies with the elastic limit, and is readily determined by plotting the intersection of Euler's curve and a horizontal line representing the elastic limit.

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The following theoretical view of the failure of long and moderately long columns of structural steel or iron, accords with the experiments.

All columns being assumed to possess initial eccentricity, a small finite quantity, then:

For frictionless pin-ended, round-ended, or knife-edge-ended columns the formula $\frac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$ is correct, but only applicable so long as $E$ remains constant. If now the modulus of compression changed slowly at and above elastic limit, we might introduce it as a variable into Euler's equation and obtain a curve tangenting the primary curve at the elastic limit, and lying entirely to the left of it. But in this paper it has been shown that at this point $E$ becomes zero, and remains at zero during change of state of the whole, or of that portion of the column whose shortening causes failure. In long columns there is but a small part of the metal that ever passes elastic limit. Making $E$ in the formula $= 0$, gives an absurd result. The formula therefore fails entirely. So long as $E$ remains $= 0$ resistance cannot increase. When the column happens to retain its straightness until there is again a modulus of compression, it would be permissible to reintroduce it and obtain a new formula for specimens which behave that way. But the experiments show that this does not occur with any certainty; very seldom will it occur when $\frac{l}{r}$ is above 42. Below that length-ratio we are not discussing; entirely different conditions of the metal prevail, and the strength of such short pillars has plainly no relation to that of longer ones.

When the column has any degree of fixity of ends (including ordinary pin as well as flat and fixed-ended columns) the effect of such fixity may be treated as diminishing its effective length considered as a frictionless hinge-ended column, according to the usual reasoning, and because elastic reactions of metal are frictionless, therefore the resistances should be represented by $a \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$, $a$ being greater than unity. But by reason of the statical moment exerted by the ends upon the bearings, and perhaps even in a greater degree depending upon the size and character of those bearings, some load lower than the product of elastic limit by cross-section causes a part or the whole of the cross-section at the ends to reach elastic limit and suffer change of state. This is accompanied by unstable equilibrium (see foot-note, page 59), and the column is at once in the condition of either a frictionless hinge-ended one of a length equal its whole length, or intermediate between its former virtual length and its whole length, according as the fixing moment disappears more or less completely. This new longer length may be such that the load already on is in excess of its appropriate strength and sudden failure ensues.

This explanation seems to fit the series of 3-inch square bars. The Watertown detailed reports show that at from 150 000 to 180 000 pounds load the scale started in front of pins, which means of course that the elastic limit of that portion of metal was passed. At $\frac{l}{r} = 138$, the belt of results reverses its curvature, and the total loads on the two closely agreeing specimens of that length are 183 500 pounds and 180 000 pounds. At $\frac{l}{r} = 159$, the total loads on specimens of that length are

154 150 and 160 500, and from thence to $\frac{l}{r} = 194$, the total loads of 11 specimens out of 12, range but from 147 180 to 160 500, or but 3 000 pounds per square inch on projected intrados of pin-bearings. It seems as if the adjustments of these bars were so perfect that some part of the metal had to pass elastic limit in order to allow the bar to fail at all, and that they otherwise unanimously refused to obey Euler's law up to $\frac{l}{r} = 194$.

Sudden failure may also be caused when the end-fixing moment depends for existence solely upon pin-friction, by the friction of quiescence becoming overmatched by the end-moment. This is believed to have been the case of the writer's experiments with 1-inch squares where the pins were lubricated. The end-bearings of these bars were the full section of the bar, therefore in cases of absolutely perfect adjustment a greater percentage of the full elastic limit is obtainable than if the bearings had been smaller, because the ends will not pass elastic limit with so small a load. Compare two tests with deviation .00 at $\frac{l}{r} = 132$ and 138, diagram for 1-inch square iron with upper limit of belt of 3-inch square tests.

# TABLE No. 1.

Compression Tests of Steel Bars. All from same Blow of Bessemer Steel, "as from Rolls."—Testing Laboratory of Cambria Iron Company, May, 1886. C. A. Marshall, Engineer of Tests.

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Laboratory number. | Dimensions of cross-section. Inches. | Length. Inches. | Length. Diameters. $\frac{l}{r}$ | | Condition of piece before test (all being full size as from rolls). | Elastic limit compression. Pounds per square inch. | Elastic limit. Tension. See Table No. 2. | Load per square inch at failure. | Second maximum. Pounds per square inch. | Load per square inch at removal. | Modulus of elasticity. Pounds per square inch. | Permanent shortening. Per cent. | Permanent deflection. Inches. | Permanent shortening with 100 000 pounds per square inch. Per cent. | Remarks. |
| 1667a | ¾ round | 1.5 | 2 | .... | | 47 300 | 46 030 | ..... | ..... | ..... | ..... | .... | .... | 14.8 | |
| 1667 | ¾ " | 9.37 | 12.5 | 50 | | ..... | | 44 580 | 45 580 | ..... | 27 880 000 | .... | .09 | .... | Gave way 3 inches from end. |
| 1669a | 1 " | 2 | 2 | .... | | 46 170 | | ..... | ..... | ..... | ..... | .... | .... | 14.8 | Under 200 000 lbs. per sq. in. original, shortening = 47.0 per cent. This equals 99 550 lbs per sq in. actual enlarged area at center. |
| 1669b | 1 " | 2 | 2 | .... | | 45 870 | 44 202 | ..... | ..... | ..... | ..... | .... | .... | 14.9 | |
| 1669 | 1 " | 12 | 12 | 48 | Gentle curve, .03 inch ordinate central | .... | | 44 000 | 44 000 | 33 410 | 27 590 000 | 1.25 | .38 | .... | Flexed orthodox. |
| 1551a | 1¼ " | 2.5 | 2 | .... | | 42 970 | 40 747 | ..... | ..... | ..... | ..... | ... | .... | 15.2 | |
| 1551b | 1¼ " | 2 5 | 2 | .... | | 43 950 | | ..... | ..... | ..... | ..... | .... | .... | 15.2 | |
| 1551 | 1¼ " | 15 | 12 | 48 | Bent, .035 inch ordinate 5 inches from end | ..... | | 40 850 | None. | 22 060 | ..... | 2.67 | 1.06 | ... | Flexed orthodox, ends scaled first. |
| 1544a | 1½ " | 3 | 2 | .... | | 41 720 | 40 275 | ..... | ..... | ..... | ..... | .... | .... | 15.4 | |
| 1544b | 1½ " | 3 | 2 | .... | | 40 860 | | ..... | ..... | ..... | ..... | .... | .... | 15.3 | |
| 1544 | 1½ " | 18 | 12 | 48 | Straight | ..... | | 41 880 | None. | 28 870 | ..... | 1.78 | .78 | | |
| 1543a | 1¾ " | 3 5 | 2 | .... | | 42 870 | 40 017 | ..... | ..... | ..... | ..... | .... | .... | 15.3 | |
| 1543b | 1¾ " | 3.5 | 2 | .... | | 41 280 | | ..... | ..... | ..... | ..... | .... | .... | 15.3 | |
| 1543 | 1¾ " | 21 | 12 | 48 | | ..... | | 39 950 | 39 910 | 31 690 | ..... | 1.52 | .71 | | |
| 1539a | 2 " | 4 | 2 | .... | | 40 620 | 38 207 | | | | | | | | |
| 1539b | 2 " | 4 | 2 | .... | One end shows bruise of shear blade | 37 040 | | | | | | | | | |
| 1539 | 2 " | 24 | 12 | 48 | | ..... | | 40 850 | None. | 31 990 | ..... | 1.29 | .74 | .... | Flexed orthodox. |
| 1535a | 2¼ " | 4½ | 2 | .... | | 38 300 | ..... | | | | | | | | |
| 1535b | 2¼ " | 4½ | 2 | .... | | 37 950 | | | | | | | | | |
| 1535 | 2¼ " | 27 | 12 | 48 | | ..... | | 36 790 | 37 130 | 33 050 | ..... | 1.59 | .80 | | |
| 1534 | 2¼ " | 40.5 | 18 | 72 | | ..... | | 36 580 | None. | 25 500 | ..... | .69 | | | |
| 1530a | 2½ " | 5 | 2 | .... | | 35 740 | 36 100 | | | | | | | | |
| 1530b | 2½ " | 5 | 2 | .... | | 37 940 | | | | | | | | | |
| 1530 | 2½ " | 30 | 12 | 48 | | ..... | | 35 650 | None. | 32 650 | ..... | 1.10 | .75 | .... | Gave way, ends and middle together. |
| 1531 | 2½ " | 44 | 17.6 | 70 | Shows bruise and bend caused by shear blade at one end | ..... | | 34 450 | None. | 26 270 | ..... | .48 | .89 | | |
| 1595a | ¾ square | 1½ | 2 | .... | | 44 290 | 44 273 | ..... | ..... | ..... | ..... | .... | .... | 14.6 | |
| 1595b | ¾ " | 1½ | 2 | .... | | 43 400 | | ..... | ..... | ..... | ..... | .... | .... | 14.4 | |
| 1595 | ¾ " | 9.37 | 12.5 | 45 | | ..... | | 44 960 | 44 210 | 37 490 | 27 740 000 | 1.92 | .28 | .... | Flexed orthodox. |
| 1591a | $\frac{5}{16}$ " | 1.6 | 2 | .... | | 49 750 | 47 815 | ..... | ..... | ..... | ..... | .... | .... | 13.9 | |
| 1591b | $\frac{5}{16}$ " | 1.6 | 2 | .... | | 48 360 | | ..... | ..... | ..... | ..... | .... | .... | 14.0 | |
| 1587a | 1 " | 2 | 2 | .... | | 44 025 | | ..... | ..... | ..... | ..... | .... | .... | 14.9 | |
| 1587b | 1 " | 2 | 2 | .... | | Lost. | 43 560 | ..... | ..... | ..... | ..... | .... | .... | 14.5 | Under 200 000 pounds per square inch original, shortening = 45.3 per cent. |
| 1587 | 1 " | 12 | 12 | 42 | | ..... | | 43 080 | 44 710 | 40 120 | 28 950 000 | 3.21 | .41 | | |
| 1583a | 1¼ " | 2.5 | 2 | .... | | 41 040 | | ..... | ..... | ..... | ..... | .... | .... | 13. 5 | |
| 1583b | 1¼ " | 2.5 | 2 | .... | | 43 560 | 41 060 | ..... | ..... | ..... | ..... | .... | .... | 13. 4 | |
| 1583 | 1¼ " | 15 | 12 | 42 | Bent, .05 inch ordinate 6 inches from end | ..... | | 40 060 | 37 690 | 36 360 | 28 920 000 | 2.13 | .59 | .... | Flexure started in opposite directions at the two ends, at finish is orthodox. |
| 1579a | 1½ " | 3 | 2 | .... | | 42 550 | 39 317 | ..... | ..... | ..... | ..... | .... | .... | 13.4 | |
| 1579b | 1½ " | 3 | 2 | .... | | 42 930 | | ..... | ..... | ..... | ..... | .... | .... | 13.4 | |
| 1579 | 1½ " | 18 | 12 | 42 | | ..... | | 38 420 | 38 920 | 35 810 | ..... | 1.72 | .59 | | |
| 1575a | 1¾ " | 3.5 | 2 | .... | | 40 610 | 38 193 | | | | | | | | |
| 1575b | 1¾ " | 3.5 | 2 | .... | | 40 650 | | | | | | | | | |
| 1575 | 1¾ " | 21 | 12 | 42 | | ..... | | 39 450 | None. | 33 780 | ..... | 2.24 | .93 | .... | Flexed orthodox, very slowly. |
| 1570a | 2 " | 4 | 2 | .... | | 39 870 | | | | | | | | | |
| 1570 | 2 " | 24 | 12 | 42 | | ..... | | 39 750 | 39 650 | 38 340 | ..... | 2.05 | .97 | | |
| 1567a | 2¼ " | 4½ | 2 | .... | | 40 240 | 38 310 | | | | | | | | |
| 1567b | 2¼ " | 4½ | 2 | .... | | 39 640 | | | | | | | | | |
| 1567 | 2¼ " | 27 | 12 | 42 | | ..... | | 39 270 | None. | 34 620 | ..... | 0.48 | .32 | | |
| 1566 | 2¼ " | 40.5 | 18 | 63 | | ..... | | 37 330 | " | 28 550 | ..... | 0.54 | .85 | .... | Resistance diminished very slowly and gradually. |
| 1453 | 3 × $\frac{3}{8}$ " | 4.5 | 12 | 42 | Heavy hot stamped figures in middle | ..... | 47 363 | 47 650 | " | 45 090 | ..... | 0.67 | .04 | | |
| 1453a | " " | 6.75 | 18 | 63 | | ..... | | 46 420 | " | 38 020 | ..... | 0.32 | .07 | | |
| 1453b | " " | 9 | 24 | 83 | | ..... | | 46 150 | " | 34 480 | ..... | | .10 | .... | Flexed orthodox. |
| 1457 | 3 × ½ " | 6 | 12 | 42 | Heavy hot stamped figures | ..... | | 41 650 | ..... | ..... | ..... | 1.17 | .14 | .... | Not held squarely. At 33 050 pounds per square inch had perceptibly deflected. Behavior similar to 1583. |
| 1457a | " | 12 | 24 | 83 | | ..... | 44 417 | 43 490 | ..... | ..... | ..... | 0.25 | .26 | | |
| 1457b | " " | 15 | 30 | 104 | | ..... | | 42 170 | ..... | ..... | ..... | 0.20 | .28 | .... | Flexed orthodox. Just before failure deflection = .02 inch. |
| 1461 | 3 × ¼ " | 9 | 12 | 42 | | ..... | 41 447 | 43 580 | 43 350 | 43 350 | ..... | 2.00 | .20 | .... | Scaled from top downward. |
| 1461a | " " | 22 | 29.3 | 102 | Crooked. Maximum ordinate .02 inch | ..... | | 40 700 | None. | 26 970 | ..... | 0.18 | .28 | .... | Flexed orthodox. |
| 1465 | 3 × 1 " | 12 | 12 | 42 | Straight | ..... | 39 397 | 41 200 | 40 610 | 40 610 | ..... | 1.83 | .28 | .... | " |
| 1465a | " " | 24 | 24 | 83 | Crooked. Maximum ordinate .05 inch | ..... | | 36 920 | None. | 26 600 | ..... | 0.29 | .37 | .... | Just before failure deflection = .02 inch. |
| 1469 | 3 × 1¼ " | 14 | 11.2 | 39 | Straight | ..... | 38 482 | 38 350 | 42 980 | 42 610 | ..... | 2.22 | .35 | | |
| 1469a | " " | 22.5 | 18 | 63 | Gentle curve, .09 inch, ordinate central | ..... | | 35 430 | None. | 31 300 | ..... | 0.13 | .20 | .... | Flexed orthodox. Just before failure deflection = .03 inch. |
| 1473 | 3 × 1½ " | 18 | 12 | 42 | " .03 " " | ..... | 37 820 | 36 920 | 37 520 | 36 790 | ..... | 2.11 | .62 | .... | Flexed orthodox. No perceptible deflection before failure. |
| 1477 | 3 × 1¾ " | 21 | 12 | 42 | Straight | ..... | 35 917 | 35 760 | 37 580 | 37 160 | ..... | 1.81 | .63 | ... | Just before failure deflection = .005 inch. |
| 1481 | 3 × 2 " | 24 | 12 | 42 | " | ..... | 39 302 | 37 670 | ..... | 33 740 | ..... | 0.46 | .23 | .... | Flexed orthodox. |
| 1488 | 4 × ½ " | 6 | 12 | 42 | " | ..... | | 55 420 | ..... | 52 360 | ..... | 2.17 | .12 | .... | Flexed orthodox. At load 50 970 per square inch specimen calipered, and found to have recovered length perfectly. |
| 1488a | " " | 9 | 18 | 63 | " | ..... | 53 800 | 55 110 | ..... | 52 950 | ..... | 0.44 | .02 | | |
| 1488b | " " | 12 | 24 | 83 | " | ..... | | 55 320 | ..... | 39 520 | ..... | 0.33 | .18 | .... | Gave way rather suddenly. |
| 1488c | " " | 15 | 30 | 104 | Heavy stamped figures across bar 2 inches from one end, causing a sharp bend, .02 inch ordinate | ..... | | 51 210 | ..... | 13 340 | ..... | 0.13 | .60 | .... | Gave way very suddenly 5.5 inches from end. |
| 1497 | 4 × 1 " | 12 | 12 | 42 | Straight | ..... | 41 415 | 42 630 | 42 310 | 41 570 | ..... | 2.50 | .45 | | |
| 1497a | " " | 30 | 30 | 104 | Slight bend, ordinate .02 inch, near end | ..... | | 39 270 | None. | 24 890 | ..... | .... | .... | .... | Just before failure deflection = .06 inch. Failed suddenly orthodox. |
| 1500 | 4 × 1¼ " | 15 | 12 | 42 | | ..... | 36 680 | 39 300 | 38 700 | 38 700 | ..... | 1.73 | .46 | .... | No perceptible deflection before failure. |
| 1500a | " " | 30 | 24 | 83 | Gentle curve, .04 inch ordinate | ..... | | 38 200 | None. | 27 530 | ..... | 0.20 | .40 | .... | Gave way suddenly, orthodox. Just before failure deflection = .01 inch. |
| 1503 | 4 × 1½ " | 18 | 12 | 42 | Straight | ..... | 37 580 | 39 500 | 40 640. | 40 640 | ..... | 2.05 | .45 | .... | No perceptible deflection before failure. |
| 1503a | " " | 26.9 | 18 | 63 | Slight bend, ordinate .02 inch, at 6 inches from end | ..... | | 39 850 | None. | 32 920 | ..... | 0.33 | .27 | .... | Gave way first at the bend. |
| 1503 | " " | 45 | 30 | 104 | Slight bend, ordinate .025 inch, at 6 inches from end | ..... | | 39 100 | None. | 32 920 | ..... | 0.11 | .70 | .... | Just before failure deflection = .03 inch. |

# TABLE No. 2.

Tensile Tests of Steel Bars. Pieces Cut from Same Bars used in Compression Tests, Table No. 1.—Testing Laboratory of Cambria Iron Company May, 1886. C. A. Marshall, Engineer of Tests.

| No. of tests in average. | Dimensions of cross-section. Inches. | Elastic limit. Lbs. per sq. in. | Ultimate strength. Lbs. per sq. in. | Length on which elongation is given. Inches. | Per cent. elongation. | Per cent. reduction of area. | Modulus of elasticity. Lbs. per sq. in. | Character of fracture. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|
| 4 | ¾ round. | 46 090 | 68 995 | 8 | 26.3 | 48.2 | 28 980 000 | Silky and dull. | One test only for modulus. |
| 4 | 1 " | 44 202 | 67 970 | 10 | 25.7 | 41.5 | ..... | Dull crystalline and dull. | |
| 3 | 1¼ " | 40 747 | 67 040 | 12 | 26.8 | 46.9 | ..... | Dull. | |
| 3 | 1½ " | 40 275 | 66 363 | 15 | 25.8 | 41.9 | ..... | Dull crystalline and dull. | |
| 3 | 1¾ " | 40 017 | 66 333 | 18 | 24.0 | 33.6 | ..... | Two silky, one dull crystalline and dull. | |
| 3 | 2 " | 38 207 | 65 663 | 20 | 23.9 | 27.8 | ..... | One silky, two crystalline. | |
| 1 | 2¼ " | 37 000 | 65 460 | 22 | 19.9 | 17.2 | ..... | Crystalline. | |
| 1 | 2½ " | 36 100 | ..... | 25 | 10.2 | .... | ..... | ..... | Not broken with 61 180 pounds per square inch. |
| 3 | ¾ square. | 44 273 | 68 427 | 8 | 26.5 | 42.5 | ..... | Silky, some dull crystalline. | |
| 2 | 9/16 " | 47 815 | 69 390 | 8 | 24.9 | 38.9 | ..... | Silky and dull. | |
| 3 | 1 " | 43 560 | 68 510 | 10 | 25.6 | 37.1 | ..... | Silky and dull crystalline. | |
| 3 | 1¼ " | 41 060 | 67 973 | 12 | 26.0 | 39.2 | ..... | Dull and dull crystalline. | |
| 3 | 1½ " | 39 317 | 66 833 | 15 | 25.1 | 36.8 | ..... | Silky, some crystalline. | |
| 3 | 1¾ " | 38 193 | 66 400 | 18 | 24.6 | 33.0 | ..... | Crystalline, dull center. | |
| 1 | 2¼ " | 38 310 | ..... | 22 | 7.6 | .... | ..... | ..... | Not broken with 60 050 pounds per square inch. |
| 3 | 3 × ⅜ | 47 363 | 68 657 | 15 | 27.0 | 44.0 | 29 350 000<br>29 640 000 | Silky. | Two tests for modulus on strips. |
| 3 | 3 × ½ | 44 417 | 67 527 | 15 | 26.5 | 42.8 | ..... | 90 per cent. silky, 10 per cent. faint crystalline. | |
| 3 | 3 × ¾ | 41 447 | 66 987 | 18 | 25.6 | 39.1 | ..... | 75 per cent. silky, 25 per cent faint crystalline. | |
| 3 | 3 × 1 | 39 397 | 66 700 | 18 | 24.9 | 35.9 | ..... | 40 per cent. silky, 60 per cent. dull crystalline. | |
| 4 | ½ × 1¼ | 38 482 | 66 640 | 8 | 26.4 | 45.9 | ..... | Silky and dull. | Strips cut from 3 × 1¼-inch bar. |
| 4 | ½ × 1½ | 37 820 | 66 342 | 8 | 21.7 | 31.1 | 29 410 000<br>29 860 000 | Silky and crystalline. | " 3 × 1½ " 2 tests for modulus. |
| 4 | 7/16 × 1¾ | 35 917 | 65 762 | 8 | 27.6 | 44.1 | ..... | Three silky, one part crystalline. | " 3 × 1¾ " |
| 4 | ⅜ × 2 | 39 302 | 66 537 | 8 | 26.4 | 46.6 | ..... | Silky. | " 3 × 2 " |
| 2 | 4 × ½ | 53 800 | 71 255 | 20 | 21.0 | 36.9 | ..... | Silky. | Bar finished at very low heat. |
| 3 | 4 × ¾ | 41 527 | 66 917 | 20 | 24.5 | 35.1 | ..... | 25 per cent. silky, 75 per cent. crystalline. | |
| 2 | 4 × 1 | 41 415 | 66 230 | 22 | 24.3 | 36.5 | ..... | 25 per cent. silky, 75 per cent. crystalline. | |
| 1 | 4 × 1¼ | 36 680 | ..... | .. | .... | .... | ..... | ..... | Not broken. Beyond capacity of machine. |
| 1 | 4 × 1½ | 37 580 | ..... | .. | .... | .... | ..... | ..... | " " " |

All tested full size as from rolls except where noted.

# TABLE No. 3.

Compression Tests of Iron Bars.—Testing Laboratory of Cambria Iron Company, June, 1886. C. A. Marshall, Engineer of Tests.

| 1. Laboratory No. | 2. Dimensions of cross-section. Inches. | 3. Length. Inches. | 4. Length. Diameters. | 5 $\frac{l}{r}$ | 6. Condition of piece before test. | 7. Elastic limit compression, Lbs. per square inch. | 8. Elastic limit tension. See Table 4 | 9. Load per square inch at failure. | 10. Second maximum. Pounds per square inch | 11. Load per square inch at removal. | 12. Modulus of elasticity, Pounds per square inch. | 13. Permanent shortening. Per cent. | 14. Permanent deflection. Inches. | 15. Permanent shortening with 100-000 lbs per sq. in. Per cent. | 16. Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2158 | $\frac{3}{4}$ round.... | 9.5 | 12.7 | 51 | Fairly straight..... | ..... | 35 530 | 36 750 | 35 640 | 32 300 | 26 160 000 | 1.3 | .28 | ..... | Bent orthodox. |
| 2159 | $\frac{3}{4}$ " .... | 9.5 | 12.7 | 51 | " " ..... | ..... | | 36 750 | 36 970 | 30 070 | 26 240 000 | 1.5 | .35 | ..... | " |
| 2162 | 1 " .... | 12 | 12 | 48 | " " ..... | ..... | | 36 210 | 37 270 | 37 270 | 26 440 000 | 0.83 | .25 | ..... | " |
| 2163 | 1 " .... | 12 | 12 | 48 | " " ..... | ..... | | 36 580 | 37 370 | 37 370 | 26 350 000 | 0.75 | .22 | ..... | " |
| 2166 | 1 " .... | 30 | 30 | 120 | Sharp bend 5 inches from end; ordinate at that point = .08..... | ..... | 36 170 | 31 580 | None. | 18 460 | ..... | 0.10 | .30 | ..... | Failed at the bend. |
| 2164 | 1 " .... | 2 | 2 | .... | ..... | 36 890 | | ..... | ..... | ..... | ..... | .... | ... | 20.25 | At 100 000 lbs. per sq. in. Specimen skewed. |
| 2196 | $\frac{3}{4}$ square.... | 9.5 | 12.7 | 44 | Fairly straight..... | ..... | 32 975 | 34 590 | 34 590 | 33 870 | 27 420 000 | 2.0 | .28 | ..... | Bent orthodox. |
| 2197 | $\frac{3}{4}$ " .... | 9.5 | 12.7 | 44 | " " ..... | ..... | | 32 790 | 31 520 | 30 780 | 27 790 000 | 1.7 | .30 | ..... | " |
| 2168 | 1 " ... | 12 | 12 | 42 | " " ..... | ..... | 32 800 | 33 250 | 36 500 | 36 500 | 27 300 000 | 1.4 | .22 | ..... | " |
| 2169 | 1 " .... | 12 | 12 | 42 | " " ..... | ..... | | 33 550 | 34 720 | 34 720 | 27 990 000 | 1.33 | .29 | ..... | " |
| 2172 | 1 " .... | 30 | 30 | 104 | Gentle curve .03 inch, ordinate central..... | ..... | | 30 680 | None, | 18 750 | ..... | 0.16 | .35 | ..... | Failed orthodox in direction of initial bend. |
| 2174 | $1\frac{1}{2}$ " ... | 18 | 12 | 42 | Fairly straight..... | ..... | 25 560 | 27 290 | 30 140 | 30 140 | ..... .... | 1.3 | .48 | ..... | |
| 2175 | $1\frac{1}{2}$ " .... | 27 | 18 | 62 | Straight..... | ..... | | 25 970 | None. | 22 080 | ..... | 0.8 | .70 | ..... | Failed orthodox. |
| 2177 | 2 " .... | 24 | 12 | 42 | Gentle curve, .03 inch, ordinate central..... | ..... | 26 930 | 27 170 | 31 360 | 31 360 | ..... | 1.3 | .62 | ..... | Bent orthodox. |
| 2214 | $3 \times \frac{1}{2}$..... | 6 | 12 | 42 | Straight..... | ..... | 28 165 | 30 070 | 38 040 | 38 040 | ..... | 1.8 | .11 | ..... | " |
| 2215 | " ..... | 12 | 24 | 83 | " ..... | ..... | | 28 360 | None. | 19 610 | .... .... | 0.6 | .30 | ..... | " |
| 2217 | " ... .... | 15 | 30 | 104 | Fairly straight..... | ..... | | 27 710 | None. | 19 610 | ..... | 0.3 | .22 | ..... | " |
| 2219 | " ..... | 18 | 36 | 125 | Ordinate .02 inch, along middle 10 inches..... | ..... | | 27 640 | None. | 19 610 | ..... | 0.1 | .18 | ..... | " |
| 2191 | $2\frac{15}{16} \times 1\frac{1}{4}$..... | 15 | 12 | 42 | Straight..... | ..... | 26 470 | 27 570 | 31 940 | 31 940 | ..... | 2.3 | .28 | ..... | " |
| 2192 | " ..... | 30 | 24 | 83 | Gentle curve, .04 inch, ordinate central..... | .. .. | | 26 510 | None. | 20 560 | ..... | 0.23 | .34 | ..... | " |
| 2194 | $3.2 \times 1\frac{3}{4}$..... | 21 | 12 | 42 | Straight..... | ..... | 27 430 | 27 700 | 32 570 | 32 570 | ..... | 1.6 | .43 | ..... | " |

## TABLE No. 4.

Tensile Tests of Iron Bars—Testing Laboratory of Cambria Iron Company, June, 1886.
C. A. Marshall, Engineer of Tests.

| Laboratory No. | Dimensions of cross-section. In. | Elastic limit. Lbs. per sq. in. | Ultimate Strength. Lbs. per sq. in. | Length on which elongation is given. | Per cent. elongation. | Per cent. reduction of area. | Modulus of elasticity. Lbs. per sq. in. | Character of fracture. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|
| 2157 | ¾ round. | 35 640 | 55 060 | 8 | 19.5 | 28.3 | 27 500 000 | Dark silky. | |
| 2160 | ¾ " | 35 420 | 54 620 | 8 | 20.4 | 29.1 | 27 410 000 | " | |
| 2161 | 1 " | 35 520 | 53 400 | 10 | 12.4* | 19.0 | 26 700 000 | Bright short fibrous. | *Not including fracture. Broke at an injury near grip. |
| 2165 | 1 " | 36 820 | 56 300 | 10 | 20.0 | 26.4 | 27 540 000 | Bright short fibrous. Specks granular. | |
| 2195 | ¾ square. | 32 610 | 49 930 | 8 | 27.7 | 35.9 | 28 990 000 | Dark silky. | |
| 2198 | ¾ " | 33 340 | 49 270 | 8 | 21.6 | 35.8 | 29 180 000 | " | |
| 2167 | 1 " | 32 550 | 51 220 | 10 | 26.4 | 32.0 | 27 900 000 | " | |
| 2171 | 1 " | 33 050 | 51 340 | 10 | 27.4 | 41.5 | 28 290 000 | " | |
| 2173 | 1½ " | 25 560 | 47 020 | 15 | 25.5 | 34.4 | ..... | Laminated fibrous. | |
| 2176 | 2 " | 26 930 | 47 520 | 18 | 15.1 | 20.3 | ..... | Banded fibrous, 5 per cent. granular. | |
| 2213 | 3 × ½ | 27 970 | 49 220 | 15 | 19.4 | 27.7 | ..... | Short fibrous. | |
| 2218 | 3 × ½ | 28 360 | 49 480 | 15 | 22.8 | 28.4 | ..... | " | |
| 2190 | 2 15/16 × 1¼ | 26 470 | 47 530 | 20 | 24.1 | 27.1 | ..... | Laminated, dark fibrous | |
| 2193 | 3.2 × 1¾ | 27 430 | 43 850 | 24 | 7.8 | 12.9 | ..... | Coarse fibrous, sliver on edge. | |

Many of the bars are rough. All tested full size as from rolls.

## TABLE No. 7.

Compression Tests of Steel and Iron Bars 1 inch square by 12 inches long. Bars marked W all from same blow of 70 000-pound Cambria Bessemer Steel. Bars marked K all from same lot of Union Iron Mills Iron.

| Flat or hinged ends | Bar mark. | Elastic limit. Lbs. per sq. in. | Maximum resistance. Lbs. per sq. inch. | Load at removal. | Chord shortening. Per cent. | Deflection. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | | | Inch. | |
| Flat | W 1 | 49 720 | 50 220 | 40 000 | 2.66 | .36 | Deflection orthodox. |
| " | W 2 | 48 400 | 49 200 | 40 000 | 2.83 | .39 | " |
| " | W 3 | 49 070 | 49 070 | 45 000 | 1.42 | .10 | " |
| " | W 4 | 49 700 | 51 790 | 45 000 | 2.66 | .36 | Bent in reverse curve. |
| " | W 5 | 47 880 | 47 880 | 47 280 | 1.00 | .26 | Scale disturbed at ends first, removed as soon as scale much disturbed in middle. |
| " | W 6 | 50 200 | 50 790 | 45 000 | 1.50 | .16 | Deflection orthodox. |
| " | W 7 | 48 170 | 49 260 | 45 000 | 2.17 | .14 | |
| " | W 9 | 50 170 | 51 160 | 45 000 | 1.66 | .20 | |
| | | 49 164 | 49 921 | ..... | .... | .... | Average. |
| Hinged | W 5 | 51 350 | 51 350 | 45 000 | 2.75 | .09 | |
| " | W 8 | 47 480 | 48 470 | 36 000 | 1.33 | .32 | |
| | | 49 415 | 49 910 | ..... | .... | .... | Average. |
| Flat | K 1 | 35 100 | 38 220 | 38 220 | 1.66 | .21 | |
| " | K 2 | 35 600 | 38 270 | 38 270 | 1.25 | .24 | |
| " | K 3 | 35 500 | 39 500 | 39 400 | 1.50 | .20 | |
| " | K 5 | 34 340 | 38 150 | 38 150 | 1.50 | .18 | |
| " | K 6 | 36 410 | 38 780 | 38 780 | 1.33 | .25 | |
| " | K 7 | 33 680 | 35 730 | 35 730 | 1.33 | .25 | |
| " | K 8 | 31 760 | 36 840 | 36 840 | 1.50 | .19 | |
| " | K 9 | 36 000 | 39 400 | 39 400 | 1.42 | .20 | |
| Hinged | K 11 | 33 380 | 37 650 | 37 650 | 1.66 | .17 | Bent along axis. |
| Flat | K 13 | 32 520 | ..... | 32 520 | 0.66 | .03 | When scaled all along. |
| | | ..... | 36 220 | 36 220 | 1.58 | .20 | |
| " | K 14 | 33 860 | 37 440 | 37 440 | 1.42 | .21 | |
| " | K 15 | 33 960 | 36 430 | 36 430 | 1.25 | .22 | |
| " | K 16 | 34 410 | ..... | 34 410 | 0.66 | .04 | When scaled all along. |
| | | ..... | 36 630 | 36 630 | 1.42 | .20 | |
| " | K 17 | 34 180 | 36 200 | 36 200 | 1.33 | .23 | |
| " | K 18 | 34 750 | ..... | 34 750 | 0.66 | .09 | When scaled all along in 5 minutes. |
| | | ..... | 37 560 | 37 560 | 1.17 | .24 | |
| " | K 19 | 32 300 | ..... | 32 300 | 0.75 | .12 | When scaled all along. |
| | | ..... | 34 000 | 34 000 | 1.42 | .24 | |
| " | K 20 | 33 400 | 36 340 | 36 340 | 1.50 | .25 | |
| | | 34 185 | 37 256 | ..... | .... | .... | Average. |
| Hinged | K 4 | 32 900 | 32 900 | 32 000 | .... | .15 | |
| " | K 10 | 34 180 | 36 000 | 36 000 | 1.08 | .16 | |
| " | K 12 | 34 240 | 37 840 | 37 840 | 1.50 | .22 | |
| | | 33 773 | 35 580 | ..... | .... | .... | Average. |

# TABLE No. 5.

Compression Tests of Miscellaneous Bars. – Testing Laboratory of Cambria Iron Company, 1886. C. A. Marshall, Engineer of Tests.

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Laboratory No. | Dimensions of cross-section. Inches. | Length. Inches. | Elastic limit compression. Lbs per sq. in. | Elastic limit tension. Lbs. per sq. in. | Load per sq. in. at failure. Lbs. | Second maximum. Lbs. per sq. in. | Load per sq. in. at removal. Lbs. | Modulus of elasticity. Lbs. per sq. in. | Permanent shortening. Per cent. | Permanent deflection. Inches. | Permanent Shortening with 100 000 pounds per sq. in. Per cent. | Kind of material. | Remarks. |
| 331 | 1 round..... | 12 | .... | 41 230 | 42 000 | None. | 30 000 | 29 740 000 | 2.0 | .50 | ..... | Open hearth steel, all from same melt. | |
| 330 | 1 " ..... | 12 | .... | | 42 480 | 43 120 | 30 000 | ..... | 1.8 | .46 | ..... | | |
| 348 | $\frac{8}{16}$ square..... | 9.62 | .... | 42 830 | 43 160 | 42 000 | 30 000 | 29 010 000 | 3.5 | .58 | ..... | | |
| 352 | $\frac{8}{16}$ " ..... | 9.62 | .... | | 43 900 | None. | 30 000 | ..... | 2.8 | .60 | ..... | | |
| 350 | 1 square ..... | 12 | .... | 39 570 | 39 990 | 38 910 | 30 000 | ..... | 3.1 | .74 | ..... | | Gave way at a bend 2 inches from middle. |
| 358 | 1 " ..... | 12 | .... | | 40 390 | 40 080 | 30 000 | ..... | 3.2 | .64 | ..... | | |
| 359 | .799 r'd turned from .8 square | 2.39 | 42 000 | 42 830 | .... | .... | .... | ..... | .. | .. | .1587 | | Maximum stress per square inch original = 113 200.<br>" " actual = 89 460. |
| 457 | $\frac{3}{4}$ round..... | 9.37 | .... | 54 430 | 53 960 | 50 380 | 50 380 | 29 420 000 | 0.9 | .20 | ..... | Open hearth steel, all from same melt. | Gave way at injury by clutch. |
| 454 | $\frac{3}{4}$ " ..... | 9.37 | .... | | 53 130 | 49 210 | 49 210 | ..... | 0.9 | .21 | ..... | | " " |
| 476 | 1 round..... | 12 | .... | 50 830 | 51 500 | 49 920 | 49 920 | 29 420 000 | 1.0 | .28 | ..... | | |
| 479 | 1 " ..... | 12 | .... | | 49 500 | 50 270 | 50 270 | 29 200 000 | 1.0 | .28 | ..... | | |
| 466 | $\frac{8}{16}$ square..... | 9.62 | .... | 51 810 | 51 000 | 59 980 | 59 980 | 28 670 000 | 1.7 | .23 | ..... | | |
| 503 | $\frac{8}{16}$ " ..... | 9.62 | .... | | 51 250 | 57 650 | 57 650 | ..... | 2.0 | .34 | ..... | | |
| 500 | 1 square..... | 12 | .... | 49 260 | 48 240 | 57 340 | 57 340 | ..... | 2.0 | .45 | ..... | | |
| 506 | 1 " ..... | 12 | .... | | 48 340 | 58 080 | 58 080 | ..... | 1.8 | .39 | ..... | | |
| 510 | 94 r'd turned from 1 round. | 2.84 | 50 000 | 50 830 | .... | ... | .... | ..... | .. | .. | .0426 | | Maximum stress per square inch, original = 168 700.<br>" " actual = 126 350. |
| 573 | $\frac{8}{16}$ square..... | 9.62 | 68 000 | 69 000 | .... | 88 720 | 88 720 | 29 220 000 | 1.0 | .20 | ..... | Open hearth steel, all from same melt. | Elastic limits by micrometer. Beam showed limit of 573 at 73 260 lbs., and of 579 at 72 000 lbs., but there was no failure to sustain those loads. There was but one maximum corresponding to second maximum as given for the other materials. |
| 579 | $\frac{8}{16}$ " ..... | 9.62 | 69 000 | | .... | 89 420 | 89 420 | 29 350 000 | 1.0 | .24 | ..... | | |
| 559 | 1 round..... | 12 | 67 000 | 67 110 | .... | 79 110 | 79 110 | 29 330 000 | 0.8 | .21 | ..... | | Elastic limits by micrometer. Beam failed to show limit. But one maximum. |
| 547 | 1 " ..... | 12 | 67 500 | | .... | 79 110 | 79 110 | 29 200 000 | 0.75 | .22 | ..... | Spring steel. | |
| 547a | .94 r'd turned from 1 round. | 2.83 | 70 000 | | .... | .... | .... | ..... | .. | .. | .0065 | | Maximum stress per square inch, original = 190 000.<br>" " actual = 166 030. |
| 2336a | 1 round..... | 12 | 49 500 | 49 750 | .... | .... | .... | ..... | 2.54 | .61 | ..... | Open hearth steel. | |
| 2336b | .94 r'd turned from 1 round. | 2.84 | 48 000 | | 49 500 | 48 000 | 35 000 | 28 830 000 | .. | .. | .0985 | | Maximum stress per square inch, original = 125 000.<br>" " actual = 99 360. |
| 2348a | 1 round..... | 12 | .... | 32 670 | 30 220 | 30 090 | 22 800 | ..... | 2.5 | .35 | ..... | O. H. steel. | |
| 2351a | 1 " ..... | 12 | .... | 44 090 | 43 690 | None. | 35 000 | ..... | 1.2 | .31 | ..... | " | |
| 2352a | 1 " ..... | 12 | .... | 43 250 | 44 000 | 40 000 | 40 000 | 30 490 000 | 1.0 | .15 | ..... | " | |
| 2353a | 1 " ..... | 12 | .... | 46 000 | 45 750 | None. | 35 000 | ..... | 1.6 | .40 | ..... | " | |
| 2350a | 1 " ..... | 12 | .... | 48 160 | 47 520 | 46 390 | 35 000 | ..... | 2.5 | .57 | ..... | " | |
| 2335a | 1 " ..... | 12 | .... | 50 600 | 49 020 | 49 270 | 40 000 | ..... | 1.7 | .36 | ..... | " | |
| 2343a | 1 " ..... | 12 | .... | 32 010 | 32 650 | 33 650 | 30 000 | ..... | 1.6 | .11 | ..... | Bessemer steel. | |
| 2344a | 1 " ..... | 12 | .... | 36 920 | 37 820 | None. | 30 000 | ..... | 1.5 | .36 | ..... | " | |
| 2347a | 1 " ..... | 12 | .... | 39 460 | 41 510 | " | 35 000 | ..... | 1.2 | .24 | ..... | " | |
| 2346a | 1 " ..... | 12 | .... | 39 570 | 41 720 | 40 460 | 35 000 | ..... | 1.2 | .17 | ..... | " | |
| 2345a | 1 " ..... | 12 | .... | 40 000 | 39 500 | None. | 30 000 | 29 790 000 | 1.8 | .42 | ..... | " | |
| 2338a | 1 " ..... | 12 | .... | 47 780 | 46 260 | " | 35 000 | ..... | 1.4 | .42 | ..... | " | |
| 2337a | 1 " ..... | 12 | .... | 47 000 | 45 500 | " | 35 000 | 29 810 000 | 1.6 | .45 | ..... | " | |
| 2293 | 1 " ..... | 12 | .... | 47 600 | 48 500 | 42 810 | 35 000 | 29 240 000 | 1.8 | .46 | ..... | " | |
| 2307 | 1 " ..... | 12 | .... | 39 000 | 40 000 | 36 600 | 25 000 | 28 570 000 | 2.3 | .72 | ..... | Iron rolled from scrap. | |
| 2309 | 1 " ..... | 12 | .... | 36 000 | 38 000 | 36 500 | 25 000 | 28 570 000 | 2.2 | .58 | ..... | | |
| 2311 | 1 " ..... | 12 | .... | 42 780 | 38 500 | 35 580 | 25 000 | 28 180 000 | 1.7 | .56 | ..... | | |
| 2309a | .94 r'd turned from 1 round. | 2.83 | 38 000 | 36 000 | .... | .... | .... | ..... | .... | .... | ..... | | Maximum stress per square inch, original = 91 710.<br>" " actual = 73 850. |
| 2391a | ⅞ round..... | 9.06 | .... | 35 500 | 36 500 | 36 940 | 30 000 | 29 910 000 | 4.0 | .46 | ..... | Sligo stay-bolt iron. | |
| 2391b | .80 r'd turned from ⅞ round | 3.5 | 36 000 | | 38 000 | 48 910 | 48 910 | 30 120 000 | 9.0 | .23 | ..... | | |
| 2391c | .80 r'd turned from ⅞ round | 2.49 | 34 000 | | | | | | | | | | Maximum stress per square inch, original = 80 000.<br>" " actual = 67 070. |

# TABLE No. 6.

Tensile Tests of Miscellaneous Bars.—Testing Laboratory of Cambria Iron Company. C. A. Marshall, Engineer of Tests.

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Laboratory No. | Lab. Nos. of compression tests from same lot. | Dimensions of cross-section. | Elastic limit. Lbs. per sq. in. | Ultimate strength. Lbs. per sq. in. | Gauged length. | Elongation. per cent. | Reduction of area per cent. | Modulus of Elasticity. Lbs per sq. in. | Appearance of fracture. | Kind of material. | REMARKS. |
| | | Inch. | | | In. | | | | | | |
| 329 | 330, 331 | 1 round. | 41 230 | 67 320 | 10 | 28.7 | 59.4 | 30 420 000 | Fine silky..... | O. H. steel from same melt. | |
| 346 | 348, 352, 350 | $\frac{5}{16}$ square. | 42 830 | 67 500 | 8 | 29.5 | 57.0 | 29 850 000 | Cup, fine silky..... | | |
| 357 | 358, 359 | 1 square. | 39 570 | 66 120 | 10 | 27.8 | 57.4 | ..... | Silky..... | | |
| 455 | 457, 454 | $\frac{3}{4}$ round. | 54 430 | 102 300 | 8 | 19.4 | 30.9 | 29 150 000 | Fine crystal, dull center. | O. H. Steel from same melt. | |
| 484 | 476, 479, 466 | 1 " | 50 830 | 101 900 | 10 | 14.9 | 25.4 | 29 500 000 | " " " spot... | | |
| 497 | 503, 500 | $\frac{5}{16}$ square. | 51 810 | 102 200 | 8 | 17.2 | 28.9 | 29 640 000 | Crystal, dull spot..... | | |
| 512 | 506, 510 | 1 " | 49 260 | 101 600 | 10 | 14.0 | 18.8 | 29 960 000 | Crystalline..... | | |
| 576 | 573, 579 | $\frac{3}{8}$ " | 74 510 | 144 400 | 8 | 7.2 | 10.2 | 29 420 000 | " ..... | O. H. steel from same melt. | Elastic limit by beam and flaking of scale |
| 574 | | $\frac{3}{16}$ " | 69 000 | 144 700 | 8 | 7.0 | 8.0 | 29 200 000 | " ..... | | " " micrometer. |
| 549 | 559, 547, 547a | 1 round. | 67 110 | 144 600 | 10 | 9.3 | 10.7 | 29 580 000 | " ..... | | " " " beam failed to show limit. |
| 2336 | 2336a—b | 1 " | 49 750 | 80 825 | 10 | 27.0 | 47.0 | 29 630 000 | Dull crystal, dull spot.... | O H. steel. | |
| 2348 | 2348a | 1 " | 32 670 | 45 980 | 10 | 34.1 | 69.6 | ..... | Half cup, silky..... | " | |
| 2351 | 2351a | 1 " | 44 690 | 71 400 | 10 | 26.2 | 56.0 | ..... | Cup, silky..... | " | |
| 2352 | 2352a | 1 " | 43 250 | 74 625 | 10 | 25.0 | 55.3 | 29 960 000 | " " ..... | " | Each from a different melt. |
| 2353 | 2353a | 1 " | 46 000 | 78 375 | 10 | 25.4 | 52.4 | ..... | Silky and dull..... | " | |
| 2350 | 2350a | 1 " | 48 160 | 82 150 | 10 | 21.2 | 47.8 | ..... | Dull..... | " | |
| 2335 | 2335a | 1 " | 50 660 | 82 430 | 10 | 22 4 | 52.2 | ..... | Silky and dull..... | " | |
| 2343 | 2343a | 1 " | 32 010 | 52 100 | 10 | 33.6 | 64.6 | ..... | " ..... | Bessemer steel. | |
| 2344 | 2344a | 1 " | 36 920 | 61 110 | 10 | 27.6 | 63.4 | ..... | " ..... | " | |
| 2347 | 2347a | 1 " | 39 460 | 61 870 | 10 | 29.4 | 60.1 | ..... | " and dull..... | " | |
| 2346 | 2346a | 1 " | 39 570 | 63 140 | 10 | 29.2 | 54 2 | ..... | " ..... | " | |
| 2345 | 2345a | 1 " | 40 000 | 64 680 | 10 | 27.5 | 58.0 | 30 420 000 | " and dull..... | " | Each from a different heat. |
| 2338 | 2338a | 1 " | 47 780 | 72 050 | 10 | 18.1 | — | ..... | Dull, some faint crystal... | " | Broke in grip. |
| 2337 | 2337a | 1 " | 47 000 | 74 625 | 10 | 23.2 | 46.0 | 30 370 000 | " " crystal, dull center.. | " | |
| 2292 | 2293 | 1 " | 47 600 | 74 650 | 10 | 23.3 | 52.1 | ..... | " " " " .. | " | |
| 2306 | 2307 | 1 " | 39 000 | 56 810 | 10 | 21.0 | 30.0 | 28 570 000 | Coarse silky..... | Iron rolled from scrap. | |
| 2308 | 2309 | 1 " | 36 000 | 51 790 | 10 | 21.8 | 53.8 | 28 480 000 | Dark " ..... | | |
| 2310 | 2311 | 1 " | 41 000 | 57 690 | 10 | 15.2 | 38.4 | 28 480 000 | " " ..... | | |
| 2310 | 2311 | 1 " | 42 780 | | | | | | | | Elastic limit by micrometer. |
| 2390 | 2391a-b-c | 1 " | 35 500 | 55 890 | 9 | 29.7 | 54.4 | 30 190 000 | Half cup, fine silky..... | Sligo stay-belt iron. | " " beam. |

# TABLE No. 8.

## Compression Tests of 1-inch Square Steel Bars, Hinged.

(*See Plate X.*)

All from same blow, distinguished by letter W. Tested as from rolls with rocking bearings, bars vertical, axis of pins east and west. All dimensions in inches; loads in pounds per square inch. "Deviation" is from straight line through centers of ends of bars at the middle of length.

| Piece mark and Rod number. | Length. | $\frac{l}{r}$ ratio. | Condition before Test. | | | Elastic limit from Table No. 7. | Maximum resistance. | Difference maximum—Elastic limit. | Principal deviation just before failure. | Just after failure. | | Load removed. Deviation. | Maximum resistance upon reapplying load. | Maximum shortening per inch. | Total chord shortening. | Remarks. Mode of failure, etc. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Twist, Arc for radius 1. | Deviation towards N. or S. | Deviation towards E. or W. | | | | | Load. | Deviation. | | | | | |
| W 9· | 16 | 55 | ..... | 00 | 00 | 50 170 | 49 380 | —790 | ..... | 41 720 | ..... | N. .30 | ..... | ..... | .14 | Not removed till load fallen to 30 000. |
| W 9: | 16 | 55 | ..... | 00 | 00 | 50 170 | 50 170 | 000 | ..... | 39 730 | ..... | N. .30 | ..... | ..... | .22 | " " |
| W 9· | 20 | 69 | ..... | 00 | 00 | 50 170 | 50 720 | +550 | ..... | 32 690 | ..... | S. .31 | ..... | ..... | .195 | Rather sudden at 6 inches from upper end. |
| W 9: | 20 | 69 | ..... | 00 | W. .01 | 50 170 | 51 110 | +940 | ..... | 19 810 | ..... | S. .30 | ..... | ..... | ·055 | Slowly to 38 720; then suddenly to 19 810. |
| W 9· | 22 | 76 | ..... | S. .03 | W. .02 | 50 170 | 48 380 | −1 790 | ..... | 26 830 | ..... | S. .31 | ..... | ..... | .035 | Sudden. |
| W 9: | 22 | 76 | ..... | S. .04 | W. .04 | 50 170 | 45 300 | —4 870 | ..... | 21 850 | ..... | S. .36 | ..... | ..... | .04 | " |
| W 5· | 24 | 83 | ..... | .00 | E. .01 | 49 610 | 49 060 | —550 | ..... | 19 900 | ..... | S. .44 | ..... | .01 | .05 | " Elastic limit is mean of two tests. |
| W 5: | 24 | 83 | ..... | N. .02 | W. .01 | 49 610 | 48 950 | —660 | ..... | 19 900 | ..... | N. .44 | ..... | .015 | .05 | " |
| W 8· | 26 | 90 | ..... | S. .02 | 00 | 47 480 | 46 950 | —530 | ..... | 18 820 | ..... | S. .45 | ..... | .012 | .05 | " |
| W 8: | 26 | 90 | ..... | 00 | W. .02 | 47 480 | 49 260 | +1 780 | ..... | 18 820 | ..... | S. .45 | ..... | ..... | .05 | " after standing a while at maximum. |
| W 8· | 28 | 97 | ..... | N. .02 | E. .02 | 47 480 | 47 720 | +240 | ..... | 33 000 | ..... | N. .20<br>E. .75 | ..... | .025 | .08 | " bent along axis. |
| W 8: | 28 | 97 | ..... | N. .01 | W. .02 | 47 4 0 | 46 480 | —1 000 | ..... | 17 580 | ..... | S. .57 | ..... | .007 | .04 | " |
| W 2 | 30 | 104 | ..... | S. .02 | W. .02 | 48 400 | 47 670 | —1 730 | ..... | 13 930 | ..... | N. .57 | ..... | .01 | .06 | " |
| W 2: | 30 | 104 | ..... | S. .02 | 00 | 48 400 | 45 370 | —3 030 | ..... | 15 120 | ..... | S. .62 | ..... | .015 | .04 | " |
| W 8· | 32 | 111 | ..... | 00 | E. .02 | 47 480 | 46 360 | —1 080 | ..... | 16 450 | ..... | S. .56 | ..... | .01 | ·04 | Very sudden. |
| W 8: | 32 | 111 | .02 | N. .02 | W. .03 | 47 480 | 45 650 | —1 830 | N. .08 | 15 900 | N. > .98 | N. .60 | ..... | .02 | .04 | |
| W 6 | 34 | 118 | ..... | 00 | 00 | 50 200 | 45 560 | —4 640 | ..... | 15 780 | ..... | S. .54 | ..... | .01 | .03 | Very sudden. |
| W 6: | 34 | 118 | ..... | N. .03 | W. .02 | 50 200 | 44 620 | —5 580 | N. .10 | 15 800 | N. .89 | N. .64 | ..... | .015 | ·04 | |
| W 8· | 36 | 125 | .10 | S. .025 | W. .02 | 47 480 | 34 660 | —12 820 | ..... | 15 100 | ..... | S. .32 | ..... | ..... | .01 | |
| W 8: | 36 | 125 | Some. | N. .06 | E. .03 | 47 480 | 31 680 | —15 800 | N. .15 | 16 000 | N. .74 | N. .28 | ..... | .005 | .01 | |
| W 7. | 38 | 132 | ..... | S. .01 | W. .03 | 48 170 | 41 690 | —6 480 | N. .11 | 13 600 | N. > .90 | N. .43 | ..... | .01 | .02 | |
| W 7: | 38 | 132 | ..... | N. .015 | W. .03 | 48 170 | 43 280 | —4 890 | ..... | 12 960 | ..... | E. .52<br>N. .32 | ..... | .01 | .07 | Bent east first, then north. |
| W 8· | 40 | 138 | ..... | S. .015 | W. .03 | 48 170 | 37 980 | —10 190 | S. .045 | 13 000 | S. .97 | S. .43 | ..... | 00 | .01 | Very sudden. Scale disturbed only near middle. |
| W 5· | 40 | 138 | ..... | N. .08 | E. .015 | 49 610 | 19 210 | —32 140 | N. .15 | 15 900 | N. .59 | N. .12 | 15 400 | 00 | 00 | Scale disturbed middle 12 inches. |
| W 7· | 42 | 145 | .06 | 00 | W. .01 | 48 170 | 34 000 | —14 170 | S. .03 | 13 600 | S. .85 | S. .23 | >11 800 | .004 | 00 | Maximum lengthening per inch middle S. = .005. |
| W 7: | 42 | 145 | ..... | N. .05 | 00 | 48 170 | 16 600 | —31 570 | S. .10 | 14 600 | S. .39 | S. .05 | 15 800 | 00 | 00 | Scale disturbed in middle 12 inches by reapplication. |
| W 7· | 44 | 152 | .06 | N. .06 | W. .02 | 48 170 | 26 000 | —22 170 | N. .12 | 12 100 | N. .82 | N. .16 | 12 700 | .002 | 00 | Scale disturbed in middle by reapplication. |
| W 7: | 44 | 152 | ..... | S .025 | E. .01 | 48 170 | 30 000 | —18 170 | S. .06 | 12 200 | S. .87 | S. .16 | >12 000 | 00 | 00 | Scale slightly disturbed N. side middle 15 inches. |
| W 6· | 48 | 166 | ..... | N. .02 | W. .03 | 50 200 | 13 130 | —37 070 | N. .40 | 11 540 | N. .52 | N. .02 | 13 980 | ..... | ..... | Scale not disturbed. |
| Same shifted on bearings .02 south of axis..... | | | | | | 50 200 | 22 870 | —27 330 | N. .08 | 11 440 | N. .77 | N. .04 | 16 810 | 00 | 00 | Scale slightly disturbed middle 6 inches S. side. |
| W 6: | 48 | 166 | .05 | N. .06 | E. .04 | 50 200 | 10 660 | —39 540 | N. .92 | 10 660 | N. .92 | N. .15 | 9 850 | 00 | 00 | Scale disturbed middle 14 inches by reapplication. |
| W 9· | 52 | 180 | ..... | N. .04 | W. .02 | 50 170 | 9 810 | ..... | N. .11 | 9 210 | N .42 | N. .04 | 9 510 | 00 | 00 | Scale not disturbed. |
| Same shifted on bearings .02 south of axis..... | | | | | | 50 170 | 13 270 | ..... | N. .13 | 9 810 | N. .54 | N. .05 | ..... | 00 | 00 | " |
| " .04 " ..... | | | | | | 50 170 | 18 810 | ..... | N. .13 | 9 810 | N. .79 | N. .05 | ..... | 00 | 00 | " |
| " .06 " ..... | | | | | | 50 170 | 20 600 | ..... | N. .04<br>W. .06 | 9 710 | S. .72 | N. .04 | 13 870 | 00 | 00 | " |
| W 9: | 52 | 180 | ..... | 00 | E. .02 | 50 170 | 12 420 | ..... | N. .07 | 9 830 | N. .42 | 00 | ..... | ..... | ..... | Scale not disturbed. |
| Same applying pressure at start to cause failure in opposite direction | | | | | | ..... | 29 810 | ..... | S. .08 | 10 830 | S. 1.06 | S. .09 | 9 730 | .002 | 00 | Scale slightly disturbed middle 14 inches north side. |
| W 6· | 56 | 194 | ..... | S. .09 | E. .03 | 50 200 | 8 400 | ..... | S. .54 | 8 000 | S. .56 | S. .11 | ..... | ..... | ..... | Scale not disturbed. |
| Same shifted on bearings .09 north of axis..... | | | | | | 50 200 | 8 800 | ..... | S. .03 | 8 200 | N. .21 | S. .08 | ..... | ..... | ..... | " |
| " .06 " ..... | | | | | | 50 200 | 18 900 | ..... | S. .16 | 8 700 | S. .94 | S. .09 | ..... | ..... | ..... | " |
| " .07 " ..... | | | | | | 50 200 | 20 000 | ..... | S. .22 | 8 800 | S. .99 | S. .09 | ..... | 00 | 00 | " |
| W 4· | 60 | 208 | ..... | S .06 | E. .03 | 49 700 | 8 000 | ..... | S. .08 | 7 200 | S. .41 | S. .17 | 8 000 | ..... | ..... | " |
| Same shifted on bearings .06 north of axis..... | | | | | | 49 700 | 10 000 | ..... | S. .02 | 7 300 | N. 47 | S. .04 | ..... | ..... | ..... | " |
| " .04 " ..... | | | | | | ..... | 17 900 | ..... | S. .11 | 7 300 | S. .97 | S. .07 | ..... | 00 | 00 | " |

# TABLE No. 9.

## Compression Tests of 1-inch Square Iron Bars, Hinged.

(*See Plate XI.*)

All from same lot of iron distinguished by letter K. Tested as from rolls with rocking bearings, bars vertical, axis of pins east and west. All dimensions in inches; loads in pounds per square inch. "Deviation" is from straight line through centers of ends of bars at middle of length.

| Piece mark and rod number. | Length. | $\frac{l}{r}$ ratio. | Condition before test. | | | Elastic limit from Table No. 7. | Maximum resistance. | Difference maximum — elastic limit. | Principal deviation just before failure. | Just after failure. | | Load removed. Deviation. | Maximum resistance upon reapplying load. | Maximum shortening per inch. | Total chord shortening. | Remarks. Mode of failure, etc. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Twist. Arc for radius 1. | Deviation towards N. or S. | Deviation towards E or W. | | | | | Load. | Deviation. | | | | | |
| K —· | 16 | 55 | .... | S. .01 | W. .01 | ..... | 34 700 | ..... | ..... | ..... | ..... | S. .. | ..... | .015 | .11 | Load at removal 22 000. |
| K —: | 16 | 55 | .... | N. .01 | W. .015 | ..... | 34 300 | ..... | W. .065 | 32 000 | W. .075 | W. .305* | ..... | .025 | .13 | Bent along axis. * After load fallen to 30 900. |
| K 1· | 20 | 69 | .... | 00 | W. .02 | 35 100 | 34 910 | — 190 | W. .10, S. .04 | 30 000 | W. .17, S. .05 | W. .44*, S .12* | ..... | .02 | .10 | " * " 25 000. |
| K 1: | 20 | 69 | .... | 00 | 00 | 35 100 | 33 910 | — 1 190 | S. .03 | 31 500 | S. .08 | S .28* | 19 000* | .02 | .04 | * After load fallen to 20 000. |
| K 1· | 22 | 76 | .... | N. .025 | 00 | 35 100 | 33 910 | — 1 190 | N. .035 | 19 940 | N. .355 | N. .365* | ..... | .015 | .04 | * " " 18 000. |
| K 1: | 22 | 76 | .04 | S. .015 | 00 | 35 100 | 34 700 | — 400 | ..... | 18 000 | S. .. | S. .34 | ..... | .015 | .04 | |
| K 11· | 24 | 83 | .... | S. .04 | W. .02 | 33 380 | 31 230 | — 2 150 | S. .09 | 16 850 | S. .40 | S. .42* | > 10 000 | .01 | .04 | * After load fallen to 16 000. |
| K 11: | 24 | 83 | .04 | N. .04 | E. .01 | 33 380 | 32 220 | — 1 160 | ..... | 17 850 | N. .. | N. .33 | ..... | .015 | .03 | |
| K 2· | 26 | 90 | .... | S. .02 | W. .02 | 35 600 | 33 750 | — 1 850 | S. .05 | 14 590 | S .56 | S. .38 | > 13 000 | .015 | .04 | |
| K 2: | 26 | 90 | .03 | S. .02 | W. .02 | 35 600 | 33 450 | — 2 150 | ..... | 16 870 | S. .. | W. .24, S. .30 | ..... | .015 | .05 | Bent west first, load falling to 26 200, then south suddenly, load falling to 16 870. |
| K 3· | 28 | 97 | .... | S. .02 | E .05 | 35 500 | 32 000 | — 3 500 | E. .09, S. .08 | 14 000 | E. .25, S. .61 | S. .41 | > 13 000 | .015 | .05 | Started to bend east, then veered south. |
| K 3: | 28 | 97 | .... | N. .06 | W. .04 | 35 500 | 30 700 | — 4 800 | N. .095 | 13 900 | N. .63 | N. .42 | > 13 400 | .015 | .03 | Sudden. |
| K 5· | 30 | 104 | .06 | S. .06 | 00 | 34 340 | 29 270 | — 5 060 | S. .12 | 12 780 | S. .65 | S. .44 | 11 710 | .01 | .05 | |
| K 18· | 30 | 104 | .... | N. .03 | W. .02 | 34 750 | 33 000 | — 1 750 | N. .05 | 12 000 | N. .73 | N. .50 | > 11 000 | .01 | .03 | |
| K 3· | 32 | 111 | .... | N. .04 | E. .02 | 35 500 | 31 350 | — 4 150 | ..... | 12 940 | N. .. | W. .40, N. .30 | ..... | .015 | .08 | Bent west first, load falling to 24 880, then north suddenly, load falling to 12 940. |
| K 3: | 32 | 111 | .06 | 00 | E. .02 | 35 500 | 34 900 | — 600 | S. .05 | 13 000 | S. .50 | E. .31, S. .26 | 13 500 | .01 | .05 | Sudden. |
| K 4· | 34 | 118 | .... | 00 | W. .04 | 32 900 | 30 940 | — 1 960 | S. .04 | 9 980 | S. .76 | S. .44 | > 9 780 | .01 | .02 | |
| K 4: | 34 | 118 | .03 | S. .05 | E. .04 | 32 900 | 26 000 | — 6 900 | S. .13 | 12 200 | S. .57 | S. .40 | 12 500 | .004 | .03 | Slow. |
| K 4· | 36 | 125 | .03 | S. .06 | W. .03 | 32 900 | 20 900 | — 12 000 | S. .13 | 13 930 | S. .53 | S. .20 | 12 920 | .01 | .01 | |
| K 4: | 36 | 125 | .... | N. .03 | W. .04 | 32 900 | 19 900 | — 13 000 | ..... | 11 140 | N. .64 | N. .30 | 11 240 | .007 | .02 | Rather slow, |
| K 5· | 38 | 132 | .... | 00 | W. .03 | 34 340 | 31 020 | — 3 320 | N. .03 | 9 170 | N. > .90 | N. .57 | > 7 000 | .01 | .04 | Sudden. |
| K 5: | 38 | 132 | .... | S. .02 | E. .02 | 34 340 | 23 320 | — 11 020 | S. .07 | 11 220 | S. .60 | S. .24 | 11 510 | .007 | .01 | " |
| K 5· | 40 | 138 | .10 | 00 | 00 | 34 340 | 30 600 | — 3 740 | N. .04 | 8 950 | N. .88 | N. .43 | > 8 950 | .005 | .01 | " |
| K 14· | 40 | 138 | .... | 00 | W. .03 | 33 860 | 18 890 | — 14 970 | N. .07 | 11 430 | N. .49 | N. .11 | 11 630 | .002 | .01 | Rather sudden. |
| K 6· | 42 | 145 | .... | 00 | W. .04 | 36 410 | 18 000 | — 18 410 | N. .04 | 12 700 | N. .43 | N. .02 | 13 000 | 00 | .01 | |
| K 6: | 42 | 145 | .... | N. .04 | E. .04 | 36 410 | 14 600 | — 21 810 | N. .09 | 12 910 | N. .39 | N. .05 | ..... | .... | .... | Very slow. Scale not disturbed. |
| Same shifted on bearings .03 south of axis | | | | | | 36 410 | 21 850 | — 14 560 | N. .11 | 10 730 | N. .74 | N. .22 | 10 630 | .002 | .01 | |
| K 7· | 44 | 152 | .... | S. .07 | 00 | 33 680 | 11 000 | ..... | S. .37 | 11 000 | S. .47 | S. .17 | 9 600 | .002 | .01 | |
| K 7: | 44 | 152 | .... | S. .03 | E. .02 | 33 680 | 28 000 | ..... | S. .08 | 8 000 | S. .97 | S. .46 | > 7 000 | .002 | .02 | |
| K 8· | 48 | 166 | .... | 00 | E. .01 | 31 760 | 14 120 | ..... | N. .. | 10 640 | N. .44 | N. .01 | ..... | .... | .... | Scale not disturbed. |
| Same shifted on bearings .01 south of axis | | | | | | 31 760 | 18 490 | ..... | N. .05 | 9 550 | N. .69 | N. .06 | 10 640 | 00 | 00 | |
| K 9· | 52 | 180 | .... | S. .05 | E .04 | 36 000 | 15 000 | ..... | S. .11 | 8 300 | S. .68 | S .08 | | | | |
| Same, applying pressure at start to cause failure in opposite direction | | | | | | ..... | 19 700 | ..... | N. .06 | 8 500 | N. .82 | N. .04 | 9 600 | | | |
| Same, applying pressure at start to cause failure towards south again | | | | | | ..... | 16 900 | ..... | S. .15 | 8 200 | S .78 | S. .09 | ..... | 00 | 00 | Scale not disturbed. |
| K. 9: | 52 | 180 | .... | S. .06 | W .05 | 36 000 | 12 000 | ..... | S. .12 | 9 100 | S .46 | S. .07 | | | | |
| Same shifted on bearings .06 north of axis | | | | | | 36 000 | 8 700 | ..... | N. .02 | 8 500 | N. .30 | S. .05 | | | | |
| " " .04 " | | | | | | 36 000 | 12 000 | ..... | S. .03 | 8 800 | N. .35 | S. .05 | | | | |
| " " .02 " | | | | | | 36 000 | 18 200 | ..... | S. .14 | 8 400 | S. .89 | S. .13 | ..... | 00 | 00 | Scale very slightly disturbed. |
| K 10· | 56 | 194 | .... | N. .04 | W. .08 | 34 180 | 8 000 | ..... | N .11 | 7 500 | N. .47 | N. .06 | 8 000 | 00 | 00 | Scale not disturbed. Very slow. |
| K 10: | 56 | 194 | .... | S. .01 | E. .07 | 34 180 | 17 500 | ..... | S. .07 | 7 600 | S .86 | S. .06 | ..... | .... | .... | Very sudden. Varying degrees of pressure at start gave maxima 17 000, 9 900, and 13 000, failures all toward south. |
| Same, applying pressure at start to cause failure in opposite direction | | | | | | ..... | 13 900 | ..... | N. .09 | 7 500 | N. .69 | N. .01 | | | | |
| K 20· | 60 | 208 | .... | S. .02 | W .04 | 33 400 | 12 800 | ..... | N. .02 | 6 400 | N. .68 | N. 00 | 9 000 | .... | .... | Rather sudden. |
| Same, applying pressure at start to cause failure in opposite direction | | | | | | ..... | 14 900 | ..... | S. .13 | 6 700 | S. .86 | S. .05 | 15 000 | | | |
| Same, applying pressure at start to cause failure towards north again | | | | | | ..... | 18 000 | ..... | S. .04 | 5 400 | N. .89 | N. .06 | ..... | .... | .... | Started to fail towards west, veered north. |

## TABLE No. 10.

### Detail of Tensile Test of 1-inch Square Steel, Series W.

Piece Mark ..... W 5
Cross Section ..... 1.000 x 1.006.
Area ..... 1.006.
Gauged Length ..... 8 and 10 inches.
Machine ..... Emery 300 000.

| Load applied. Pounds. | Stress per sq. in. Pounds. | Difference of extensions in units of .0001 inch. | Total extension Inches. | Extension per cent. | Set. Inches. | Remarks. |
|---|---|---|---|---|---|---|
| 1 006 | 1,000 | 2.50 | 000250 | .0031 | | |
| 5 030 | 5 000 | 9.50 | .001200 | .0150 | | |
| 15 090 | 15 000 | 27.25 | .003925 | .0491 | | |
| 25 150 | 25 000 | 26.75 | .006600 | .0825 | .... | Modulus of extension = 29 630 000. |
| 35 210 | 35 000 | 27.00 | .009300 | .1162 | | |
| 5 030 | 5 000 | .... | .001425 | .... | +.000225 | Modulus of elasticity E = 30 420 000. |
| 35 210 | 35 000 | .... | .009300 | .1162 | | |
| 40 240 | 40 000 | 13.75 | .010675 | .1334 | | |
| 41 246 | 41 000 | 2.25 | .010900 | .1362 | | |
| 42 252 | 42 000 | 2.50 | .011150 | .1394 | | |
| 43 258 | 43 000 | 2.75 | .011425 | .1428 | | |
| 44 264 | 44 000 | 2.50 | .011675 | .1459 | | |
| 45 270 | 45 000 | 2.75 | .011950 | .1494 | | |
| 46 276 | 46 000 | 2.50 | .012200 | .1525 | | |
| 47 282 | 47 000 | 3.75 | .012575 | .1572 | .... | Bar scaling upper end outside of gauge points. |
| 47 782 | 47 500 | 446.00 | .057175 | .7159 | .... | Scaling lower end, elastic limit by beam. |
| 47 782 | 47 500 | 250.00 | .082175 | 1.03 | .... | One minute later. |
| 47 782 | 47 500 | 975.00 | .179675 | 2.25 | .... | Two minutes later. |
| 47 782 | 47 500 | .00 | .179675 | 2.25 | .... | One minute later. |
| 47 782 | 47 500 | 10 00 | .180675 | 2.26 | .... | " |
| 47 782 | 47 500 | 7.00 | .181375 | 2.27 | .... | " |
| 48 288 | 48 000 | 8.00 | .182175 | 2.28 | .... | " |
| 49 294 | 49 000 | 12.00 | .183375 | 2.28 | .... | " |
| 50 300 | 50 000 | 240.00 | .207375 | 2.59 | .... | " |
| 51 306 | 51 000 | 109.00 | .218275 | 2.73 | .... | " |
| 52 312 | 52 000 | 229.00 | .241175 | 3.01 | | |
| 54 324 | 54 000 | 380.00 | .279175 | 3.49 | | |
| 56 336 | 56 000 | 495.00 | .328675 | 4.11 | | |
| 58 348 | 58 000 | 535.00 | .382175 | 4.78 | | |
| 60 360 | 60 000 | 640.00 | .446175 | 5.58 | .... | Micrometer removed. Above on gauged length 8 in., below on 10 in. |
| 62 372 | 62 000 | .... | 0.70 | 7.0 | | |
| 64 384 | 64 000 | .... | 0.81 | 8.1 | | |
| 66 396 | 66 000 | .... | 0.98 | 9.8 | | |
| 68 408 | 68 000 | .... | 1.18 | 11.8 | | |
| 70 420 | 70 000 | .... | 1.67 | 16.7 | | |
| 70 780 | 70 760 | .... | 2.17 | 21.7 | .... | Maximum load. Extension taken as load begins to fall. |
| 70 000 | 69 580 | .... | 2.38 | 23 8 | | |
| 67 000 | 66 600 | .... | 2.51 | 25.1 | | |
| 56 000 | 55 660 | .... | | | .... | Breaking load. |
| | | | 2.71 | 27.1 | .... | After rupture. |

Appearance of fracture, fine, silky and dull. Radial markings.
Position of fracture, 3½ inches from end mark.
Diameter, .660 square.
Per cent. reduction, 56.7.
Elongation of inch sections, .23 × .28 × .69 (fracture) × .30 ×. 26 × .21 × .20 × .19 × .18 × .17.

## TABLE No. 11.

### Detail of Compression Test of 1-inch Square Steel, Series W.

Piece Mark ..... W 5
Cross Section ..... 1.005 x 1.002.
Area ..... 1.007.
Gauged Length = .....Total = 4.020 inches.
Machine ..... Emery 300 000.

| Load applied. Pounds. | Stress per square inch original. Pounds. | Stress per square inch actual. Pounds. | Permanent compression. In. | Permanent compression per ct. | Enlarged Dimensions. | Enlarged area. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | | | | Specimen removed for measurement after each load. |
| 40 280 | 40 000 | .... | 00 | 00 | ..... | .. | Load held one minute. |
| 45 315 | 45 000 | .... | 00 | 00 | ..... | .. | " " |
| 46 322 | 46 000 | .... | .001 | 0.02 | ..... | .. | " " |
| 48 820 | 48 480 | .... | .0035 | 0.09 | ..... | .. | Elastic limit shown by beam, released load immediately. |
| 47 831 | 47 500 | .... | .006 | 0.15 | ..... | .. | Load held one minute. |
| 47 831 | 47 500 | .... | .035 | 0.87 | ..... | .. | " " |
| 47 831 | 47 500 | .... | .042 | 1.05 | ..... | .. | " " |
| 47 831 | 47 500 | .... | .050 | 1.25 | ..... | .. | " " |
| 47 831 | 47 500 | .... | .057 | 1.42 | ..... | .. | " " |
| 48 820 | 48 480 | .... | .079 | 1.97 | ..... | .. | " " |
| 48 820 | 48 480 | .... | 081 | 2.02 | ..... | .. | " " |
| 48 820 | 48 480 | .... | .082 | 2.05 | ..... | .. | " " |
| 50 350 | 50 000 | .... | .085 | 2.12 | ..... | .. | " " |
| 52 364 | 52 000 | .... | .099 | 2.47 | ..... | .. | " " |
| 54 378 | 54 000 | .... | .112 | 2.80 | ..... | .. | " " |
| 56 392 | 56 000 | 54 320 | .125 | 3.12 | 1.018×1.020 | 1.038 | " " |
| 60 420 | 60 000 | 57 520 | .155 | 3.87 | 1.026×1.024 | 1.050 | " " |
| 70 490 | 70 000 | 66 030 | .238 | 5.95 | 1.032×1.035 | 1.067 | " " |
| 80 560 | 80 000 | 73 250 | .343 | 8.57 | 1.047×1.050 | 1.099 | " " |
| 90 630 | 90 000 | 78 380 | .482 | 12.05 | 1.075×1.075 | 1.155 | " " Specimen skewed so much that it slips on face of platform. Test discontinued. |

## TABLE No. 12.

### Detail of Tensile Test of 1-inch Square Iron, Series K.

Piece Mark ..... K 11
Cross Section ..... 1.005 square.
Area ..... 1.010.
Gauged Length ..... 8 and 10 inches.
Machine ..... Emery 300 000.

| Load applied. Pounds. | Stress per square inch. Pounds. | Difference of extension in units of .0001 inch. | Total extension. Inches. | Extension per cent. | Set. Inches. | Remarks. |
|---|---|---|---|---|---|---|
| 5 050 | 5 000 | 15.00 | .001500 | .0187 | | |
| 10 100 | 10 000 | 14.00 | .002900 | .0362 | | |
| 15 150 | 15 000 | 14.75 | .004375 | .0547 | ..... | $E$ = 27 910 000 |
| 20 200 | 20 000 | 14.00 | .005775 | .0722 | | |
| 5 050 | ..... | ..... | .001475 | .... | —.000025 | |
| 25 250 | 25 000 | 14.00 | .007175 | .0897 | | |
| 26 260 | 26 000 | 3.00 | .007475 | .0934 | | |
| 27 270 | 27 000 | 3.00 | .007775 | .0972 | | |
| 28 280 | 28 000 | 2.75 | .008050 | .1006 | | |
| 29 290 | 29 000 | 4.00 | .008450 | .1056 | | |
| 30 300 | 30 000 | 3.00 | .008750 | .1094 | | |
| 31 310 | 31 000 | 6.00 | .009350 | .1169 | | |
| 32 020 | 31 700 | 1 108.00 | .120150 | 1.50 | ..... | Elastic limit by beam, held one minute. |
| 32 320 | 32 000 | 11.00 | .121250 | 1.52 | | |
| 33 330 | 33 000 | 29.00 | .124150 | 1.55 | | |
| 34 340 | 34 000 | 380.00 | .162150 | 2.03 | | |
| 35 350 | 35 000 | 180.00 | .180150 | 2.25 | | |
| 36 360 | 36 000 | 300.00 | .210150 | 2.63 | ..... | Micrometer removed. Above on gauged length of 8 inches, below on 10 inches. |
| 38 380 | 38 000 | ..... | 0.32 | 3.2 | | |
| 40 400 | 40 000 | ..... | 0.43 | 4.3 | | |
| 42 420 | 42 000 | ..... | 0 54 | 5.4 | | |
| 44 440 | 44 000 | ..... | 0.69 | 6.9 | | |
| 46 460 | 46 000 | ..... | 0.90 | 9.0 | | |
| 48 480 | 48 000 | ..... | 1.22 | 12.2 | | |
| 50 300 | 49 800 | ..... | 1.94 | 19.4 | ..... | Maximum load. |
| 46 000 | 45 540 | ..... | ..... | .... | ..... | Breaking load. Fell rapidly after maximum. |
| | | | 2.13 | 21.3 | ..... | After rupture. |

Appearance of fracture, 100 per cent. fibrous.
Position of fracture, 1½ inches from end mark.
Diameter, .835 × .840.
Per cent. reduction, 30.5.
Elongation of inch sections, .30 × .37 (fracture) × .22 × .20 × .19 × .18 × .17 × .17 × .16 × .17.

SUMMARY OF TENSILE TEST OF 1-INCH SQUARE IRON.

Series K, Piece mark K 5.

Elastic limit..... = 32 280 pounds per square inch.
Ultimate strength..... ..... = 50 490 " "
Elongation in 10 inches..... = 25.2 per cent.
Reduction of area..... = 38.0 "
Fracture fibrous..... 100 "

## TABLE No. 13.

### Detail of Compression Test of 1-inch Square Iron, Series K.

Piece Mark..... K 11
Cross Section..... .995 x .995.
Area..... .990.
Gauged Length = .....Total = 3.984.
Machine ..... Emery 300 000.

| Load applied. Pounds. | Stress per square inch original. Pounds. | Stress per square inch actual. Pounds. | Permanent shortening. Inches. | Permanent shortening per cent. | Enlarged dimensions. | Enlarged area. | Remarks. |
|---|---|---|---|---|---|---|---|
| 19 800 | 20 000 | .... | 000 | 00 | ..... | .. | Load held one minute. |
| 24 750 | 25 000 | .... | 000 | 00 | ..... | .. | " " |
| 25 740 | 26 000 | .... | 000 | 00 | ..... | .. | " " |
| 26 730 | 27 000 | .... | 000 | 00 | ..... | .. | " " |
| 27 720 | 28 000 | .... | 000 | 00 | ..... | .. | " " |
| 28 710 | 29 000 | .... | 000 | 00 | ..... | .. | " " |
| 29 700 | 30 000 | .... | .0005 | 0.01 | ..... | .. | " " |
| 30 690 | 31 000 | .... | .0015 | 0.04 | ..... | .. | " " |
| 31 680 | 32 000 | .... | .003 | 0.07 | ..... | .. | " " |
| 32 670 | 33 000 | .... | .022 | 0 55 | ..... | .. | " " |
| 32 670 | 33 000 | .... | .023 | 0.57 | ..... | .. | " four minutes. Principal elastic limit. |
| 33 660 | 34 000 | 33 830 | .021 | 0.60 | .998×.997 | .995 | Load held one minute. |
| 34 650 | 35 000 | .... | .027 | 0.67 | ..... | .. | " " |
| 35 640 | 36 000 | .... | .030 | 0.75 | ..... | .. | " " |
| 36 630 | 37 000 | .... | .032 | 0.80 | ..... | .. | " " |
| 37 620 | 38 000 | 37 690 | .039 | 0.97 | .999×.999 | .998 | " " |
| 38 610 | 39 000 | .... | .042 | 1.05 | ..... | .. | " " |
| 39 600 | 40 000 | 39 600 | .046 | 1.15 | 1.000×1.000 | 1.000 | " " |
| 41 580 | 42 000 | .... | .052 | 1.30 | ..... | .. | " " |
| 43 960 | 44 400 | 43 710 | .069 | 1.72 | 1.003×1.003 | 1.006 | " " |
| 45 540 | 46 000 | 45 190 | .082 | 2.05 | 1.004×1.004 | 1.008 | " " |
| 47 520 | 48 000 | 46 890 | .098 | 2.45 | 1.007×1.007 | 1.014 | " " |
| 49 500 | 50 000 | 48 440 | .116 | 2.90 | 1.010×1.012 | 1.022 | " " |
| 54 450 | 55 000 | 52 340 | .171 | 4.27 | 1.020×1.020 | 1.040 | " " |
| 59 400 | 60 000 | 56 240 | .236 | 5.90 | 1.028×1.028 | 1.056 | " " |
| 64 350 | 65 000 | 59 500 | .313 | 7.82 | 1.040×1.040 | 1.081 | " " |
| 69 300 | 70 000 | 62 250 | .408 | 1.02 | 1.055×1.055 | 1.114 | " " |
| 80 700 | 81 620 | .... | .756 | 1.89 | ..... | .. | Skewed $\frac{7}{16}$-inch. Test discontinued. Cross section out of square and irregular. |

## TABLE No. 14.

### Detail of Compression Test of 1-inch Round Steel, as from Rolls.

Laboratory Number..... 2336a.
Piece Mark..... 2336a.
Cross Section..... 1.007.
Area..... .800.
Gauged Length..... 8 inches central.
Total Length..... 12.015.
Machine..... Emery.

| Load applied. Pounds. | Stress per square inch. Pounds. | Difference of shortening in units of .0001 in. | Total shortening. Inches. | Shortening per cent. | Set. | Remarks. |
|---|---|---|---|---|---|---|
| 1 600 | 2 000 | 6.25 | .000625 | .0078 | | |
| 4 000 | 5 000 | 8.75 | .001500 | .0187 | | |
| 8 000 | 10 000 | 14.50 | .002950 | .0369 | ..... | |
| 12 000 | 15 000 | 14.50 | .004400 | .0550 | ..... | |
| 16 000 | 20 000 | 13.75 | 005775 | .0722 | ..... | $E$ = 28 830 000. Modulus of compression = 28 680 000. |
| 20 000 | 25 000 | 14.00 | .007175 | .0897 | ..... | |
| 24 000 | 30 000 | 13.75 | .008550 | .1069 | ..... | |
| 28 000 | 35 000 | 13.25 | .009875 | .1234 | ..... | |
| 4 000 | 5 000 | .... | ..... | .... | +.000050 | |
| 32 000 | 40 000 | 12.75 | .011150 | .1394 | | |
| 32 800 | 41 000 | 2.75 | .011425 | .1428 | | |
| 33 600 | 42 000 | 2.25 | .011650 | .1456 | | |
| 34 400 | 43 000 | 2.50 | .011900 | .1487 | ..... | First break of scale, lower end. |
| 35 200 | 44 000 | 2.25 | .012125 | .1516 | | |
| 36 000 | 45 000 | 2.50 | .012375 | .1547 | | |
| 36 800 | 46 000 | 2.25 | .012600 | .1575 | | |
| 37 600 | 47 000 | 2.50 | .012850 | .1606 | | |
| 38 400 | 48 000 | 3.00 | .013150 | .1644 | | |
| 38 800 | 48 500 | 1.25 | .013275 | .1659 | ..... | Scaled 1¼ inches lower end. |
| 39 200 | 49 000 | 3.25 | .013600 | .1700 | | |
| 39 600 | 49 500 | 5.75 | .014175 | .1772 | ..... | Going. |
| 39 600 | 49 500 | 17.50 | .015925 | .1991 | | |
| 39 600 | 49 500 | 26.00 | .018525 | .2316 | ..... | About ¾ minute intervals. |
| 39 600 | 49 500 | 29.00 | .021425 | .2678 | | |
| 39 600 | 49 500 | 34.50 | .024875 | .3109 | | |
| 39 600 | 49 500 | 65.00 | .031375 | .3922 | ..... | Scales in middle. Gone by beam. Bending left front. |
| 37 300 | 46 625 | .... | ..... | .... | ..... | Fell to this slowly. |
| 38 000 | 47 500 | .... | ..... | .... | ..... | Rose to this slowly. |
| 00 | 00 | .... | .175 | 2.19 | ..... | Removed and measured. Replaced. |
| 38 400 | 48 000 | .... | ..... | .... | ..... | Second maximum. |
| 28 000 | 35 000 | .... | ..... | .... | ..... | Load falling. Removed. Upper bearing opened at edge. |

Maximum ordinate 1 inch above middle = .61 inch.
Chord shortening..... = 0.315 "

# APPENDIX.

## CAMBRIA IRON COMPANY—TESTING LABORATORY.

JOHNSTOWN, PA., May, 1887.

DETAIL REPORTS OF COMPRESSION TESTS OF 1-INCH SQUARE STEEL, SERIES W. 70 000-POUND CAMBRIA BESSEMER STEEL, WITH HINGED ENDS.

EXPLANATIONS.

All tested as from rolls, not cold straightened.

Ends of bars were plane, and rested on hinged-rocker bearings with lubricated pins, the bearing surface of which is in plane of axes of pins.

Deviations were taken at middle of bar, and are referred to the straight line passing through centers of ends of bar. Deviations given for load 00 were measured before putting bar into testing machine by applying a straight edge. Deviations under load were obtained by measuring with a steel scale the ordinates from fine fish cords stretched parallel to bar by means of a clamp at bottom, to which one end of cord was fastened, and a similar clamp at top carrying grooved pulleys over which the cords passed with weights attached to overhanging ends. Deviations under no load and under initial load of 200 or 500 pounds are assumed to be equal.

The tests were made on a vertical 300 000-pound Emery Testing machine. Axis of pins in all cases east and west. All bars, except a few of the longest ones, were scribed every inch on all four sides, and the inches are referred to by numbers beginning at bottom.

Marked end of bar was down in all cases.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In. | In. | |
| Mark, W 5 | 00 | 00 | .. | 00 | .. | 00 | |
| | 200 | .... | .. | 00 | .. | 00 | |
| Length, 12.03 in. | 5 000 | .... | .. | 00 | .. | 00 | |
| | 10 000 | .... | .. | 00 | .. | 00 | |
| Size, | 20 000 | .... | .. | .005 | .. | 00 | |
| 1.000 x 1.005 in. | 30 000 | .... | .. | .005 | .. | 00 | |
| | 40 000 | .... | .. | .005 | .. | 00 | |
| | 41 000 | .... | .. | .005 | .. | 00 | |
| | 42 000 | .... | .. | .005 | .. | .005 | |
| | 43 000 | .... | .. | .005 | .. | .005 | |
| | 44 000 | .... | .. | .005 | .. | .005 | |
| | 45 000 | .... | .. | .005 | .. | .005 | |
| | 46 000 | .... | .. | .005 | .. | .005 | |
| | 47 000 | .... | .. | .005 | .. | .005 | |
| | 48 000 | .... | .. | .005 | .. | .005 | |
| | 49 000 | .... | .. | .005 | .. | 00 | |
| | 50 000 | .... | .. | .005 | .. | 00 | |
| | 51 000 | .... | .. | .005 | .. | 00 | Scaling on ends, slightly at middle. |
| | 51 600 | 51 350 | .. | .01 | .. | 00 | Gone. |
| | 49 800 | .... | .. | .01 | .. | 00 | Scaled 2 inches upper end, 1½ inches lower end. |
| | 49 800 | .... | .. | .01 | .01 | .. | |
| | 49 800 | .... | .. | .01 | .01 | .. | Scaled 3 inches upper end, 2 inches lower end. |
| | 50 000 | .... | .02 | .. | .03 | .. | Scaled 4 inches upper end, 3 inches lower end. |
| | 48 600 | .... | .03 | .. | .05 | .. | Scaled 5 inches upper end, 3 inches lower end. |
| | 49 400 | .... | .03 | .. | .07 | .. | Scaled 6 inches upper end, 3 inches lower end. |
| | 45 500 | .... | .. | .. | .22 | .. | |
| | 45 000 | .... | .09 | .. | .28 | .. | |
| | 200 | .... | .09 | .. | .27 | .. | |
| | 5 000 | .... | .09 | .. | .27 | .. | |
| | 10 000 | .... | .09 | .. | .27 | .. | |
| | 15 000 | .... | .09 | .. | .27 | .. | |
| | 20 000 | .... | .09 | .. | .28 | .. | |
| | 25 000 | .... | .09 | .. | .28 | .. | |
| | 30 000 | .... | .09 | .. | .28 | .. | |
| | 35 000 | .... | .09 | .. | .28 | .. | |
| | 36 000 | .... | .09 | .. | .28 | .. | |
| | 37 000 | .... | .09 | .. | .29 | .. | |
| | 38 000 | .... | .09 | .. | .29 | .. | |
| | 39 000 | .... | .09 | .. | .29 | .. | |
| | 40 000 | .... | .10 | .. | .29 | .. | |
| | 41 000 | .... | .10 | .. | .29 | .. | |
| | 42 000 | .... | .10 | .. | .29 | .. | |
| | 43 000 | .... | .10 | .. | .29 | .. | |
| | 44 000 | .... | .10 | .. | .29 | .. | |
| | 44 800 | .... | .11 | .. | .30 | .. | |
| | 43 200 | .... | .12 | .. | .32 | .. | Specimen removed. |

Chord shortening = .33 inch.

| | At 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 inches. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ordinates west | .02 | .04 | .14 | .22 | .30 | .32 | .30 | .24 | .16 | .09 | .03 inch. |
| " south | .02 | .05 | .09 | .12 | .12 | .13 | .11 | .09 | .06 | .04 | .02 " |

Shortening Per Inch.

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 inches. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| West Side | .02 | .015 | .015 | .01 | .03 | .045 | .05 | .04 | .03 | .03 | .02 | .02 inch. |
| East Side | .04 | .04 | .03 | .02 | 00 | —.01 | 00 | 00 | .02 | .03 | .05 | .05 " |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, W 8 | 00 | 00 | .. | 00 | .. | 00 | |
| | 200 | .... | .. | 00 | .. | 00 | |
| Length, 12.04 in. | 5 000 | .... | .. | 00 | .. | 00 | |
| | 10 000 | .... | .. | 00 | .. | 00 | |
| Size, | 20 000 | .... | .. | 00 | .. | 00 | |
| 1.003 x 1.004 in. | 30 000 | .... | .. | 00 | .. | 00 | |
| | 40 000 | .... | .. | 00 | .. | 00 | |
| | 41 000 | .... | .. | 00 | .. | 00 | |
| | 42 000 | .... | .. | 00 | .. | 00 | |
| | 43 000 | .... | .. | 00 | .. | 00 | |
| | 44 000 | .... | .. | 00 | .. | 00 | |
| | 45 000 | .... | .. | 00 | .. | .01 | |
| | 46 000 | .... | .. | 00 | .. | .01 | |
| | 47 000 | .... | .. | 00 | .. | .01 | Scaled 2 inches at upper end. |
| | 47 800 | 47 480 | .. | 00 | .. | .02 | Gone. |
| | 47 800 | .... | .. | 00 | .. | .01 | Scaled 2½ inches upper end, 1 inch lower end. |
| | 48 000 | .... | .. | .005 | .. | .01 | |
| | 48 800 | 48 470 | .. | .01 | .. | .02 | Scaling more rapidly. |
| | 48 000 | .... | .. | .03 | .. | .02 | Scaled 5 inches upper end, 1 inch lower end. |
| | 46 000 | .... | .. | .07 | .. | .03 | |
| | 40 000 | .... | .. | .25 | .. | .03 | Scaled 7 inches upper end, 1 inch lower end. |
| | 35 000 | .... | .. | .32 | .. | .04 | |
| | 33 000 | .... | .. | .28 | .. | .04 | Probably extent of rocking motion. |
| | 34 300 | .... | .. | .30 | .. | .04 | |
| | 36 000 | .... | .. | .32 | .. | .. | Past extent of rocking motion. |
| | 200 | .... | .. | .32 | .. | .04 | |
| | 5 000 | .... | .. | .32 | .. | .. | |
| | 10 000 | .... | .. | .33 | .. | .. | |
| | 15 000 | .... | .. | .33 | .. | .. | |
| | 25 000 | .... | .. | .. | .. | .. | Extent of rocking motion. Specimen removed. |

After Specimen Removed.

| At | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | inches. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ordinates North | .07 | .14 | .21 | .29 | .35 | .38 | .35 | .32 | .25 | .16 | .08 | .. | inch. |
| " East | .02 | .025 | .03 | .04 | ·04 | .045 | .04 | .04 | .03 | .02 | 00 | —.02 | " |

Shortening Per Inch.

| At | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | inches. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| North Side | .005 | 0 | 0 | 0 | .02 | .04 | .04 | .035 | .025 | .025 | .025 | .02 | inch. |
| East Side | .01 | 0 | 0 | 0 | 0 | 0* | 0* | 0† | .01 | .01 | .04 | .01 | " |

* Scale disturbed by tension. † Scale disturbed by compression.

| Description of Specimen. | Load. Lbs. | Load. Lbs per sq in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, W 8: | 00 | 00 | .02 | .. | .. | .03 | |
| | 200 | .... | .02 | .. | .. | .03 | |
| Length, 32.00 in. | 5 000 | .... | .02 | .. | .. | .02 | |
| | 10 000 | .... | .02 | .. | .. | .02 | |
| Size, | 15 000 | .... | .03 | .. | .. | .02 | |
| 1.003 x 1.005 in. | 20 000 | .... | .03 | .. | .. | .02 | |
| | 25 000 | .... | .03 | .. | .. | .02 | |
| Twist to right = .02 | 30 000 | .... | .03 | .. | .. | .02 | |
| inch. | 35 000 | .... | .04 | .. | .. | .02 | |
| | 36 000 | .... | .04 | .. | .. | .02 | |
| | 37 000 | .... | .04 | .. | .. | .02 | |
| | 38 000 | .... | .04 | .. | .. | .02 | |
| | 39 000 | .... | .04 | .. | .. | .02 | |
| | 40 000 | .... | .04 | .. | .. | .02 | |
| | 41 000 | .... | .05 | .. | .. | .02 | |
| | 42 000 | .... | .05 | .. | .. | .02 | |
| | 43 000 | .... | .05 | .. | .. | .02 | |
| | 44 000 | .... | .055 | .. | .. | .02 | |
| | 45 000 | .... | .06 | .. | .. | .03 | |
| | 46 000 | 45 650 | .08 | .. | .. | .05 | Goes very suddenly. |
| | 15 900 | .... | .98 | .. | .. | .. | |
| | 200 | .... | .60 | .. | .. | .01 | |
| | 5 000 | .... | .70 | .. | .. | .01 | |
| | 6 000 | .... | .74 | .. | .. | .01 | |
| | 7 000 | .... | .76 | .. | .. | .01 | |
| | 8 000 | .... | .81 | .. | .. | .02 | |
| | 9 000 | .... | .84 | .. | .. | .02 | Extent of motion. Specimen removed. |

Ordinate south at middle = .62 inch.
East and west practically straight.
Chord shortening..... = .04 inch.

Shortening Per Inch.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| South Side | 1 to 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 to 32 |
| | 0 | .01 | .01 | .02 | .02 | .01 | .005 | 0 |
| North Side | 1 | 2 to 14 | 15 | 16 | 17 | 18 to 32 | | |
| | .005 | 0 | —.005 | —.01 | —.01 | 0 | | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, W 6: | 00 | 00 | .03 | .. | .. | .02 | |
| | 200 | .... | .03 | .. | .. | .02 | |
| Length, 34.04 in. | 5 000 | .... | .04 | .. | .. | .02 | |
| | 10 000 | .... | .04 | .. | .. | .02 | |
| Size. | 15 000 | .... | .05 | .. | .. | .02 | |
| 1.006 x 1.003 in. | 20 000 | .... | .05 | .. | .. | .02 | |
| | 25 000 | .... | .05 | .. | .. | .02 | |
| | 30 000 | .... | .06 | .. | .. | .01 | |
| | 31 000 | .... | .06 | .. | .. | .01 | |
| | 32 000 | .... | .06 | .. | .. | .01 | |
| | 33 000 | .... | .07 | .. | .. | .01 | |
| | 34 000 | .... | .07 | .. | .. | .02 | |
| | 35 000 | .... | .07 | .. | .. | .02 | |
| | 36 000 | .... | .07 | .. | .. | .02 | |
| | 37 000 | .... | .07 | .. | .. | .02 | |
| | 38 000 | .... | .07 | .. | .. | .02 | |
| | 39 000 | .... | .08 | .. | .. | .02 | |
| | 40 000 | .... | .08 | .. | .. | .02 | |
| | 41 000 | .... | .08 | .. | .. | .02 | |
| | 42 000 | .... | .09 | .. | .. | .02 | |
| | 43 000 | .... | .09 | .. | .. | .02 | |
| | 44 000 | .... | .10 | .. | .. | .02 | |
| | 45 000 | 44 620 | .. | .. | .. | .. | Failed. |
| | 15 800 | .... | .89 | .. | .. | .. | |
| | 14 000 | .... | .97 | .. | .. | .. | |
| | 200 | .... | .64 | .. | .. | .02 | |
| | 5 000 | .... | .76 | .. | .. | .. | |
| | 6 000 | .... | .79 | .. | .. | .. | |
| | 7 000 | .... | .83 | .. | .. | .. | |
| | 8 000 | .... | .87 | .. | .. | .. | Extent of motion.. Specimen removed. |

Ordinate south at middle = .65 inch.
East and west practically straight.
Chord shortening..... = .04 inch.

Shortening Per Inch.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| South Side | 1 to 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 to 34 |
| | 0 | .005 | .005 | .015 | .015 | .01 | .005 | 0 |
| North Side | 1 to 16 | 17 | 18 | 19 | 20 to 34 | | | |
| | 0 | —.005 | —.005 | —.005 | 0 | | | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| Mark, W 8: | 00 | .... | .06 | .. | .03 | .. | |
| | 200 | .... | .05 | .. | .03 | .. | |
| Length, 35.97 in. | 5 000 | .... | .07 | .. | .03 | .. | |
| | 10 000 | .... | .08 | .. | .03 | .. | |
| Size, | 15 000 | .... | .085 | . | .03 | .. | |
| 1.007 x 1.003 in. | 20 000 | .... | .095 | .. | .03 | .. | |
| | 25 000 | .... | .11 | .. | .03 | .. | |
| Ends square. | 26 000 | .... | .11 | .. | .03 | .. | |
| | 27 000 | .... | .12 | .. | .03 | .. | |
| Twist to right. | 28 000 | .... | .125 | .. | .03 | .. | |
| | 29 000 | .... | .125 | .. | .03 | .. | |
| | 30 000 | .... | .13 | .. | .03 | .. | |
| | 31 000 | .... | .135 | .. | .035 | .. | |
| | 32 000 | 31 680 | .15 | .. | .04 | .. | Failed. |
| | 16 000 | .... | .74 | .. | .. | .. | |
| | 200 | .... | .28 | .. | .04 | .. | |
| | 1 000 | .... | .28 | .. | .. | .. | |
| | 2 000 | .... | .29 | .. | .. | .. | |
| | 3 000 | .... | .30 | .. | .. | .. | |
| | 4 000 | .... | .31 | .. | .. | .. | |
| | 5 000 | .... | .33 | .. | .. | .. | |
| | 6 000 | .... | .34 | .. | .. | .. | |
| | 7 000 | .... | .36 | .. | .. | .. | |
| | 8 000 | .... | .38 | .. | .. | .. | |
| | 9 000 | .... | .40 | .. | .. | .. | |
| | 10 000 | .... | .43 | .. | .. | .. | |
| | 11 000 | .... | .45 | .. | .. | .. | |
| | 12 000 | .... | .51 | .. | .. | .. | |
| | 13 000 | .... | .57 | .. | .. | .. | |
| | 14 000 | .... | .65 | .. | .. | .. | |
| | 14 500 | .... | .70 | .. | .04 | .. | Extent of motion. Specimen removed. |

Ordinate south at middle = .28 inch.
Chord shortening..... = .01 "

Shortening Per Inch.

| South Side | 1 to 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 to 36 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | .005 | .005 | .005 | .005 | .005 | .005 | .005 | .005 | .005 | .005 | 0 |

North Side.—Scale very slightly disturbed in 18 and 19 inches.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | | In. | |
| Mark, W 7. | 00 | 00 | .. | .01 | .. | .03 | |
| | 200 | .... | .. | .01 | .. | .03 | |
| Length, 37.97 in. | 5 000 | .... | .. | .01 | .. | .02 | |
| | 10 000 | .... | .. | 0 | .. | .02 | |
| Size, | 15 000 | .... | .. | 0 | .. | .02 | |
| 1.000 x 1.003 in. | 20 000 | .... | .. | 0 | .. | .02 | |
| | 25 000 | .... | .01 | .. | .. | .02 | |
| Ends square. | 30 000 | .... | .02 | .. | .. | .02 | |
| | 31 000 | .... | .02 | .. | .. | .02 | |
| | 32 000 | .... | .025 | .. | .. | .02 | |
| | 33 000 | .... | .025 | .. | .. | .02 | |
| | 34 000 | .... | .03 | .. | .. | .02 | |
| | 35 000 | .... | .03 | .. | .. | .02 | |
| | 36 000 | .... | .03 | .. | .. | .02 | |
| | 37 000 | .... | .04 | .. | .. | .02 | |
| | 38 000 | .... | .04 | .. | .. | .02 | |
| | 39 000 | .... | .05 | .. | .. | .02 | |
| | 40 000 | .... | .06 | .. | .. | .02 | |
| | 41 000 | .... | .07 | .. | .. | .03 | |
| | 41 800 | 41 690 | .11 | .. | .. | .. | Going. |
| | 13 600 | .... | > .90 | .. | .. | .. | Specimen is bent so much it throws cord out of line, and is past rocking motion. |
| | 7 300 | .... | .80 | .. | .. | .. | Extent of rocking motion. |
| | 200 | .... | .43 | .. | .. | .01 | |
| | 1 000 | .... | .44 | .. | .. | .. | |
| | 2 000 | .... | .46 | .. | .. | .. | |
| | 3 000 | .... | .49 | .. | .. | .. | |
| | 4 000 | .... | .52 | .. | .. | .. | |
| | 5 000 | .... | .55 | .. | .. | .. | |
| | 6 000 | .... | .58 | .. | .. | .. | |
| | 7 000 | .... | .63 | .. | .. | .. | |
| | 8 000 | .... | .68 | .. | .. | .. | |
| | 9 000 | .... | .74 | .. | .. | .. | |
| | 10 000 | .... | .83 | .. | .. | .. | Extent of rocking motion. Specimen removed. |

Ordinate south at middle = .48 inch.
Chord shortening..... = .02 "

Shortening Per Inch.

| South Side | 1 to 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 to 38 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | .005 | .005 | .005 | .005 | .01 | .01 | .01 | .01 | .005 | .005 | 0 |

North Side—Scale very slightly disturbed in middle three inches.

| Description of Specimen. | Load. Lbs. | Load. Lbs per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, W 8· | 00 | 00 | .. | .015 | .. | .03 | |
| | 200 | .... | .. | .015 | .. | .03 | |
| Length, 39.96 in. | 5 000 | .... | .. | .015 | .. | .01 | |
| | 10 000 | .... | .. | .015 | .. | .01 | |
| Size, | 15 000 | .... | .. | .015 | .. | .01 | |
| 1.000 x 1 000. | 20 000 | .... | .. | .015 | .. | .01 | |
| | 25 000 | .... | .. | .025 | .. | .01 | |
| | 30 000 | .... | .. | .025 | .. | 00 | |
| | 35 000 | .... | .. | .035 | .. | 00 | |
| | 36 000 | .... | .. | .045 | .. | 00 | |
| | 37 000 | .... | .. | .045 | .. | 00 | |
| | 37 980 | 37 980 | .. | .. | .. | .. | Failed very suddenly. |
| | 13 000 | 13 000 | .. | .975 | .. | .. | Scale disturbed in middle 12 inches concave side, slightly on other side. |
| | 200 | .... | .. | .435 | .. | .. | |
| | 1 000 | .... | .. | .445 | .. | .. | |
| | 2 000 | .... | .. | .465 | .. | .. | |
| | 3 000 | .... | .. | .505 | .. | .. | |
| | 3 500 | .... | .. | .515 | .. | .. | |
| | 4 000 | .... | .. | .545 | .. | .. | |
| | 4 500 | .... | .. | .555 | .. | .. | |
| | 5 000 | .... | .. | .585 | .. | .. | |
| | 5 500 | .... | .. | .605 | .. | .. | |
| | 6 000 | .... | .. | .635 | .. | .. | |
| | 7 000 | .... | .. | .705 | .. | .. | |
| | 7 500 | .... | .. | .725 | .. | .. | |
| | 8 000 | .... | .. | .765 | .. | .. | |
| | 8 500 | .... | .. | .805 | .. | .. | |
| | 9 000 | .... | .. | .845 | .. | .. | Extent of rocking motion. |
| | 200 | .... | .. | .435 | .. | .. | Removed specimen. |

Ordinate north at middle = .44 inch.
" west " = .015 "
Chord shortening..... = .01 "

Shortening Per Inch.

| | | | | | | |
|---|---|---|---|---|---|---|
| North Side | 1 to 18 | 19 | 20 | 21 | 22 | 23 to 40 |
| | 0 | .005 | .005 | .005 | .005 | 0 |

South side not perceptible.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| Mark, W 5· | 00 | 00 | .08 | .. | .015 | .. | |
| | 200 | .... | .08 | .. | .015 | .. | |
| Length, 39.98 in. | 3 000 | .... | .09 | .. | .015 | .. | |
| | 5 000 | .... | .09 | .. | .015 | .. | |
| Size, | 7 500 | .... | .10 | .. | .015 | .. | |
| 1.000 x 1.005. | 10 000 | .... | .11 | .. | .015 | .. | |
| | 12 500 | .... | .12 | .. | ·015 | .. | |
| Ends square. | 15 000 | .... | .13 | .. | .015 | .. | |
| | 17 500 | ... | .15 | .. | .015 | .. | |
| | 19 300 | 19 210 | .. | .. | .. | .. | Going. Failed. |
| | 15 900 | .... | .59 | .. | .025 | .. | |
| | 200 | .... | .12 | .. | .015 | .. | |
| | 3 000 | .... | .13 | .. | .. | .. | |
| | 5 000 | .... | .14 | .. | .. | .. | |
| | 7 500 | .... | .16 | .. | .. | .. | |
| | 10 000 | .... | .18 | .. | .. | .. | |
| | 12 500 | .... | .21 | .. | .. | .. | |
| | 14 900 | .... | .32 | .. | .. | .. | Going. |
| | 15 000 | .... | .38 | .. | .. | .. | |
| | 15 400 | .... | .48 | .. | .. | .. | |
| | 15 400 | .... | .60 | .. | .. | .. | Scales on inside. |
| | 200 | .... | .15 | .. | .025 | .. | Specimen removed. |

Ordinate south at middle = .16 inch.
Chord shortening..... = 00· "

Shortening per inch not perceptible. Scale disturbed on middle 12 inches south side.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, W 7· | 00 | 00 | 00 | 00 | .. | .01 | |
| | 200 | .... | 00 | 00 | .. | .01 | |
| Length, 41.97 in. | 5 000 | .... | .. | .01 | .. | .01 | |
| | 10 000 | .... | .. | .01 | .. | .01 | |
| Size, | 15 000 | .... | .. | .01 | .. | 00 | |
| 1.000 x 1.000. | 20 000 | .... | .. | .01 | .. | .01 | |
| | 25 000 | .... | .. | .02 | .. | .01 | |
| Twist right | 30 000 | .... | .. | .02 | .. | .01 | |
| hand = .06 in. | 31 000 | ... | .. | .02 | .. | .01 | |
| | 32 000 | .... | .. | .03 | .. | .01 | |
| | 33 000 | .... | .. | .03 | .. | .01 | |
| | 34 000 | 34 000 | .. | .. | .. | .. | Failed very suddenly. |
| | 13 600 | .... | .. | .85 | .. | .. | Extent of rocking motion. |
| | 200 | .... | .. | .23 | .. | .01 | Scale disturbed middle 14 inches north side. |
| | 1 000 | .... | .. | .24 | .. | .. | |
| | 2 000 | .... | .. | .25 | .. | .. | |
| | 3 000 | .... | .. | .27 | .. | .. | |
| | 4 000 | .... | .. | .29 | .. | .. | |
| | 5 000 | .... | .. | .31 | .. | .. | |
| | 6 000 | .... | .. | .34 | .. | .. | |
| | 7 000 | .... | .. | .38 | .. | .. | |
| | 8 000 | .... | .. | .43 | .. | .. | |
| | 9 000 | .... | .. | .50 | .. | .. | |
| | 10 000 | .... | .. | .60 | .. | .. | |
| | 11 000 | .... | .. | .73 | .. | .. | |
| | 11 800 | .... | .. | .82 | .. | .. | Extent of rocking motion. |
| | 200 | .... | .. | .23 | .. | .. | Removed specimen. |

Ordinate north at middle = .25 inch.
" East " = .01 "
Chord shortening..... = 0 "

Shortening Per Inch.

North Side { Middle 6 inches total. .025 — Middle 2 inches total. .01

South Side not perceptible.

| Description of Specimen. | Load. Lbs. | Load Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| | 00 | 00 | .05 | .. | .. | 00 | |
| Mark, W 7: | 200 | .... | .05 | .. | .. | 00 | |
| | 5 000 | .... | .04 | .. | .. | 00 | |
| Length, 41.97 in. | 10 000 | .... | .06 | .. | .01 | .. | |
| | 15 000 | .... | .09 | .. | .01 | .. | |
| Size, | 16 000 | .... | .10 | .. | .01 | .. | |
| 1.000 x 1.000. | 16 600 | 16 600 | .. | .. | .. | .. | Failed slowly. |
| | 14 600 | .... | .39 | .. | .. | .. | |
| | 14 500 | .... | .40 | .. | .. | .. | |
| | 14 500 | .... | .42 | .. | .. | .. | |
| | 200 | .... | .05 | .. | 00 | .. | Scale undisturbed. |
| | 5 000 | .... | .06 | .. | 00 | .. | |
| | 10 000 | .... | .07 | .. | 00 | .. | |
| | 12 000 | .... | .09 | .. | .01 | .. | |
| | 13 000 | .... | .10 | .. | .01 | .. | |
| | 14 000 | .... | .10 | .. | .01 | .. | |
| | 15 000 | .... | .12 | .. | .01 | .. | |
| | 15 800 | .... | .. | .. | .. | .. | Failed. |
| | 14 700 | .... | .35 | .. | .. | .. | |
| | 14 900 | .... | .54 | .. | .. | .. | |
| | 14 500 | .... | .62 | .. | .. | .. | |
| | 14 000 | .... | .69 | .. | .. | .. | Scale disturbed slightly in middle 12 inches south side. |
| | 13 500 | .... | .73 | .. | .. | .. | |
| | 200 | .... | .14 | .. | .. | .. | Removed specimen. |

Ordinate north at middle = .14 inch.
" west " = .02 "
Chord shortening..... = 00 "
Shortening per inch not perceptible where scale disturbed.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| | 00 | 00 | .06 | .. | .. | .02 | |
| Mark, W 7· | 200 | ... | .06 | .. | .. | .02 | |
| | 5 000 | .... | .06 | .. | .. | .02 | |
| Length, 43.97 in. | 10 000 | .... | .06 | .. | .. | .01 | |
| | 12 000 | .... | .07 | .. | .. | .01 | |
| Size, | 13 000 | .... | .07 | .. | .. | .01 | |
| 1.000 x 1.000 in. | 14 000 | .... | .07 | .. | .. | .01 | |
| | 15 000 | .... | .07 | .. | .. | .01 | |
| Has some dents on | 16 000 | .... | .08 | .. | .. | .01 | |
| the corners, and | 17 000 | .... | .08 | .. | .. | .02 | |
| right hand twist | 18 000 | .... | .08 | .. | .. | .02 | |
| = .06 in. | 19 000 | .... | .08 | .. | .. | .02 | |
| | 20 000 | .... | .08 | .. | .. | .02 | |
| | 200 | .... | .06 | .. | .. | .. | To confirm original deviation. |
| | 20 000 | .... | .08 | .. | .. | .02 | |
| | 21 000 | .... | .09 | .. | .. | .01 | |
| | 22 000 | .... | .09 | .. | .. | .01 | |
| | 23 000 | .... | .10 | .. | .. | .01 | |
| | 24 000 | .... | .10 | .. | .. | .01 | |
| | 25 000 | .... | .11 | .. | .. | .01 | |
| | 26 000 | 26 000 | .12 | .. | .. | .. | Failed. |
| | 12 100 | 12 100 | .82 | .. | .. | .01 | Not quite extent of rocking motion. Scale slightly disturbed in middle 12 inches south side. |
| | 200 | .... | .16 | .. | .. | .01 | |
| | 1 000 | .... | .16 | .. | .. | .. | |
| | 2 000 | .... | .16 | .. | .. | .. | |
| | 3 000 | .... | .17 | .. | .. | .. | |
| | 4 000 | .... | .18 | .. | .. | .. | |
| | 5 000 | .... | .19 | .. | .. | .. | |
| | 6 000 | .... | .20 | .. | .. | .. | |
| | 7 000 | .... | .21 | .. | .. | .. | |
| | 8 000 | .... | .22 | .. | .. | .. | |
| | 9 000 | .... | .25 | .. | .. | .. | |
| | 10 000 | .... | .28 | .. | .. | .. | |
| | 11 000 | .... | .35 | .. | .. | .. | |
| | 12 000 | .... | .51 | .. | .. | .. | Going. |
| | 12 000 | .... | .56 | .. | .. | .. | |
| | 12 400 | .... | .66 | .. | .. | .. | |
| | 12 700 | .... | .96 | .. | .. | .. | Scale disturbed in middle. |
| | 200 | .... | .24 | .. | .. | .01 | Specimen removed. |

Ordinate south at middle = .20 inch.
" west " = .01 "
Chord shortening..... = 00 "

Shortening apparently .01 inch in middle s'x inches, south side.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | | |
| Mark, W 7: | 00 | 00 | .. | .025 | .01 | .. | |
| | 200 | .... | .. | .025 | .01 | .. | |
| Length, 43.97 in. | 5 000 | .... | .. | .02 | .01 | .. | |
| | 10 000 | .... | .. | .02 | .01 | .. | |
| Size, | 15 000 | .... | .. | .03 | .01 | .. | |
| 1 000 x 1.000 in. | 20 000 | .... | .. | .03 | .01 | .. | |
| | 25 000 | .... | .. | .04 | .01 | .. | |
| No twist. | 26 000 | .... | .. | .04 | .01 | .. | |
| | 27 000 | .... | .. | .05 | .01 | .. | |
| | 28 000 | .... | .. | .05 | 01 | .. | |
| | 29 000 | .... | .. | .05 | .01 | .. | |
| | 30 000 | 30 000 | .. | .06 | .. | .. | Failed very suddenly. |
| | 12 200 | 12 200 | .. | .87 | .. | .. | Scale disturbed in middle 15 inches north side. |
| | 200 | .... | .. | .16 | 00 | .. | |
| | 1 000 | .... | .. | .16 | .. | .. | |
| | 2 000 | .... | .. | .18 | .. | .. | |
| | 3 000 | .... | .. | .19 | .. | .. | |
| | 4 000 | .... | .. | .20 | .. | .. | |
| | 5 000 | .... | .. | .21 | .. | .. | |
| | 6 000 | .... | .. | .23 | .. | .. | |
| | 7 000 | .... | .. | .25 | .. | .. | |
| | 8 000 | .... | .. | .30 | .. | .. | |
| | 9 000 | .... | .. | .36 | .. | .. | |
| | 10 000 | .... | .. | .45 | .. | .. | |
| | 11 000 | .... | .. | .63 | .. | .. | |
| | 12 000 | .... | .. | .85 | .. | .. | Extent of rocking motion. |
| | 200 | .. | .. | .17 | .. | .. | Specimen removed. |

Ordinate north at middle, = .18 inch.
" east " = .01 "
Chord shortening..... = 00 "
Shortening per inch not perceptible.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| Mark, W 6: | 00 | 00 | .06 | .. | .04 | .. | |
| | 200 | .... | .06 | .. | .04 | .. | |
| Length, 48.00 in. | 5 000 | .... | .09 | .. | .04 | .. | |
| | 6 000 | .... | .11 | .. | .03 | .. | |
| Size, | 7 000 | .... | .11 | .. | .03 | .. | |
| 1.003 x 1.003 in. | 8 000 | .... | .13 | .. | .. | .. | |
| | 8 000 | .... | .14 | .. | .03 | .. | |
| Right hand twist = | 9 000 | .... | .20 | .. | .03 | .. | |
| .05 in. | 10 000 | .... | .34 | .. | .04 | .. | |
| | 10 600 | .... | .72 | .. | .. | .. | |
| | 10 720 | 10 660 | .92 | .. | .. | .. | Failed. |
| | 200 | .... | .15 | .. | .05 | .. | |
| | 1 000 | .... | .15 | .. | .. | .. | |
| | 2 000 | .... | .17 | .. | .. | .. | |
| | 3 000 | .... | .18 | .. | . | .. | |
| | 4 000 | .... | .20 | .. | .. | .. | |
| | 5 000 | .... | .22 | .. | .. | .. | |
| | 6 000 | .... | .27 | .. | .. | .. | |
| | 7 000 | .... | .33 | .. | .. | .. | |
| | 8 000 | .... | .42 | .. | .. | .. | |
| | 9 000 | .... | .62 | .. | .. | .. | |
| | 9 900 | 9 850 | .92 | .. | .. | .. | Extent of rocking motion. Scale slightly disturbed in middle 14 inches south side, but shortening per inch not perceptible. |

Ordinate south at middle = .16 inch
" west " = .01 "
Chord shortening..... = .005 "

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, W 6 | 00 | 00 | .02 | .. | .. | .03 | |
| | 200 | .... | .02 | .. | .. | .03 | |
| Length, 48.00 in. | 5 000 | .... | .03 | .. | .. | .02 | |
| | 10 000 | .... | .04 | .. | .. | .02 | |
| Size, | 13 200 | 13 130 | .40 | .. | .. | .. | Failed. |
| 1.003 x 1.003 in. | 11 600 | 11 540 | .52 | .. | .. | .. | |
| | 200 | .... | .02 | .. | .. | .03 | |
| | 5 000 | .... | .02 | .. | .. | .02 | |
| | 10 000 | .... | .03 | .. | .. | .03 | |
| | 11 000 | .... | .04 | .. | .. | .03 | |
| | 12 000 | .... | .04 | .. | .. | .03 | |
| | 13 000 | .... | .05 | .. | .. | .02 | |
| | 14 000 | 13 980 | .06 | .. | .. | .. | Failed rather slowly. |
| | 11 500 | .... | .45 | .. | .. | .. | |
| | 200 | .... | .02 | .. | .. | .03 | |
| | 00 | .... | .02 | .. | .. | .03 | Specimen placed .02 inch south of axis. |
| | 200 | .... | .02 | .. | .. | .03 | |
| | 5 000 | .... | .02 | .. | .. | .02 | |
| | 10 000 | .... | .03 | .. | .. | .03 | |
| | 11 000 | .... | .03 | .. | .. | .03 | |
| | 12 000 | .... | .04 | .. | .. | .03 | |
| | 13 000 | .... | .04 | .. | .. | .03 | |
| | 14 000 | .... | .04 | .. | .. | .03 | |
| | 15 000 | .... | .04 | .. | .. | .03 | |
| | 16 000 | .... | .04 | .. | .. | .03 | |
| | 17 000 | .... | .05 | .. | .. | .03 | |
| | 18 000 | .... | .05 | .. | .. | .04 | |
| | 19 000 | .... | .06 | .. | .. | .04 | |
| | 20 000 | .... | .06 | .. | .. | .04 | |
| | 21 000 | .... | .06 | .. | .. | .04 | |
| | 22 000 | .... | .07 | .. | .. | .04 | |
| | 23 000 | 22 870 | .08 | .. | .. | .. | Failed suddenly. Scale slightly disturbed in middle 6 inches south side. |
| | 11 500 | 11 440 | .77 | .. | .. | .. | |
| | 200 | .... | .04 | .. | .. | .03 | |
| | 1 000 | .... | .04 | .. | .. | .03 | |
| | 2 000 | .... | .04 | .. | .. | .02 | |
| | 3 000 | .... | .04 | .. | .. | .02 | |
| | 4 000 | .... | .05 | .. | .. | .02 | |
| | 5 000 | .... | .05 | .. | .. | .02 | |
| | 6 000 | .... | .05 | .. | .. | .02 | |
| | 7 000 | .... | .05 | .. | .. | .02 | |
| | 8 000 | .... | .06 | .. | .. | .02 | |
| | 9 000 | .... | .06 | .. | .. | .02 | |
| | 10 000 | .... | .07 | .. | .. | .03 | |
| | 11 000 | .... | .07 | .. | .. | .03 | |
| | 12 000 | .... | .07 | .. | .. | .03 | |
| | 13 000 | .... | .08 | .. | .. | .03 | |
| | 14 000 | .... | .08 | .. | .. | .02 | |
| | 15 000 | .... | .09 | .. | .. | .03 | |
| | 16 000 | .... | .10 | .. | .. | .03 | |
| | 16 900 | 16 810 | .. | .. | .. | .. | Failed. |
| | 11 200 | .... | .57 | .. | .. | .. | |
| | 200 | .... | .04 | .. | .. | .03 | Specimen removed. |

Ordinate south at middle = .04 inch.
" east " = .03 "
Chord shortening..... = 00 "

Scale disturbed very slightly in middle 6 inches south side.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, W 9· | 00 | 00 | .04 | .. | .. | .02 | |
| | 200 | .... | .04 | .. | .. | .02 | |
| Length, 51.96 in. | 5 000 | .... | .06 | .. | .. | .03 | |
| | 6 000 | .... | .07 | .. | .. | .03 | |
| Size, | 7 000 | .... | .08 | .. | .. | .03 | |
| 1.008 x 1.002 in. | 8 000 | .... | .09 | .. | .. | .03 | |
| | 9 000 | .... | .11 | .. | .. | .03 | |
| | 9 900 | 9 810 | . | .. | .. | .. | Failed. |
| | 9 500 | .... | .. | .. | .. | .. | |
| | 9 300 | 9 210 | .42 | .. | .. | .. | |
| | 9 600 | .... | .. | .. | .. | .. | |
| | 9 400 | .... | .54 | .. | .. | .. | |
| | 9 600 | .... | .62 | .. | .. | .. | |
| | 9 500 | .... | .72 | .. | .. | .. | |
| | 200 | .... | .04 | .. | .. | .02 | |
| | 500 | .... | .. | .. | .. | .. | Specimen placed .02 inch south of axis. |
| | 200 | .... | .05 | .. | .. | .02 | |
| | 5 000 | .... | .07 | .. | .. | .03 | |
| | 6 000 | .... | .07 | .. | .. | .03 | |
| | 7 000 | .... | .07 | .. | .. | .03 | |
| | 8 000 | .... | .08 | .. | .. | .03 | |
| | 9 000 | .... | .09 | .. | .. | .04 | |
| | 10 000 | .... | .09 | .. | .. | .04 | |
| | 11 000 | .... | .10 | .. | .. | .04 | |
| | 12 000 | .... | .11 | .. | .. | .04 | |
| | 13 000 | .... | .13 | .. | . | .04 | |
| | 13 400 | 13 270 | .. | .. | . | .. | Failed. |
| | 9 900 | .... | .54 | .. | . | .. | Scale not disturbed. |
| | 200 | .... | .05 | .. | .. | .02 | |
| | 500 | .... | .. | .. | .. | .. | Specimen placed .04 inch south of axis. |
| | 200 | .... | .05 | .. | . | .02 | |
| | 5 000 | .... | .06 | .. | .. | .03 | |
| | 6 000 | .... | .06 | .. | .. | .03 | |
| | 7 000 | .... | .06 | .. | .. | .03 | |
| | 8 000 | .... | .065 | .. | .. | .04 | |
| | 9 000 | .... | .07 | .. | .. | .04 | |
| | 10 000 | .... | .07 | .. | .. | .04 | |
| | 11 000 | .... | .08 | .. | .. | .04 | |
| | 12 000 | .... | .08 | .. | .. | .04 | |
| | 13 000 | .... | .09 | .. | .. | .04 | |
| | 14 000 | .... | .09 | .. | .. | .04 | |
| | 15 000 | .... | .09 | .. | .. | .04 | |
| | 16 000 | .... | .10 | .. | .. | .05 | |
| | 17 000 | .... | .11 | .. | .. | .05 | |
| | 18 000 | .... | .13 | .. | .. | .06 | |
| | 19 000 | 18 810 | .14 | .. | .. | .. | Failed. |
| | 9 900 | .... | .79 | .. | .. | .. | Scale not disturbed. |
| | 200 | .... | .05 | .. | .. | .02 | |
| | 500 | .... | .. | .. | .. | .. | Specimen placed .06 inch south of axis. |
| | 200 | .... | .05 | .. | .. | .02 | |
| | 1 000 | .... | .05 | .. | .. | .03 | |
| | 5 000 | .... | .05 | .. | .. | .03 | |
| | 6 000 | .... | .05 | .. | .. | .03 | |
| | 7 000 | .... | .05 | .. | .. | .03 | |
| | 8 000 | .... | .05 | .. | .. | .04 | |
| | 9 000 | .... | .05 | .. | .. | .04 | |
| | 10 000 | .... | .05 | .. | .. | .04 | |
| | 11 000 | .... | .05 | .. | .. | .04 | |
| | 12 000 | .... | .05 | .. | .. | .04 | |
| | 13 000 | .... | .05 | .. | .. | .04 | |
| | 14 000 | .... | .05 | .. | .. | .04 | |
| | 15 000 | .... | .05 | .. | .. | .04 | |
| | 16 000 | .... | .05 | .. | .. | .05 | |

(*Mark W 9· continued.*)

| Description of Specimen. | Load. Lbs. | Load. Lbs per sq. in | Deviations. | | | | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | N. | S. | E. | W. | |
| | | | In. | | | In | |
| Mark, W 9· | 17 000 | .... | .05 | .. | .. | .05 | |
| | 18 000 | .. | .05 | .. | .. | .05 | |
| Length, 51.96 in. | 19 000 | .... | .05 | .. | .. | .06 | |
| | 20 000 | .... | .04 | .. | .. | .06 | |
| Size, | 20 800 | 20 600 | .. | .. | .. | .. | Failed. |
| 1.008 x 1.002 in. | 9 800 | .... | .72 | .. | .. | .. | |
| | 200 | .... | .04 | .. | .. | .02 | |
| | 5 000 | .... | .04 | .. | .. | .03 | |
| | 10 000 | .... | .03 | .. | .. | .04 | |
| | 11 000 | .... | .03 | .. | .. | .04 | |
| | 12 000 | .... | .02 | .. | .. | .04 | |
| | 13 000 | .... | .01 | .. | .. | .04 | |
| | 14 000 | 13 870 | .. | .. | .. | .. | Failed. |
| | 9 700 | ... | .41 | .. | .. | .. | |
| | 200 | .... | .04 | .. | .. | .02 | Specimen removed. |

Ordinate south at middle.. = .04 inch.
" east " .. = .02 "
Chord shortening..... = 0 "

Scale undisturbed.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In | | |
| Mark, W 9: | 00 | 00 | 00 | .. | .02 | .. | |
| | 500 | .... | 00 | .. | .02 | .. | |
| Length, 51.97 in. | 5 000 | .... | .01 | .. | .02 | .. | |
| | 10 000 | .... | .05 | .. | .02 | .. | |
| Size. | 11 000 | .... | .06 | .. | .02 | .. | |
| 1.005 x 1.002 in. | 12 000 | .... | .07 | .. | .02 | .. | |
| | 12 500 | 12 420 | .. | .. | .. | .. | Failed. |
| | 9 900 | 9 830 | .42 | .. | .02 | .. | |
| | 500 | .... | .01 | .. | .02 | .. | |
| | 200 | .... | 00 | .. | .02 | .. | |
| | 5 000 | .... | .. | .03 | .02 | .. | By pressure on rockers in direction to correct deviation while load rises to 5 000. |
| | 10 000 | .... | .. | .02 | .02 | .. | |
| | 11 000 | .... | .. | .02 | .02 | .. | |
| | 12 000 | .... | .. | .02 | .01 | .. | |
| | 13 000 | .... | .. | .02 | .02 | .. | |
| | 14 000 | .... | .. | .02 | .01 | .. | |
| | 15 000 | .... | .. | .02 | .01 | .. | |
| | 16 000 | .... | .. | .02 | .01 | .. | |
| | 17 000 | .... | .. | .02 | .01 | .. | |
| | 18 000 | .... | .. | .02 | .01 | .. | |
| | 19 000 | .... | .. | .02 | .01 | .. | |
| | 20 000 | .... | .. | .02 | .01 | .. | |
| | 21 000 | .... | .. | .02 | .01 | .. | |
| | 22 000 | .... | .. | .02 | .01 | .. | |
| | 23 000 | .... | .. | .02 | .01 | .. | |
| | 24 000 | .... | .. | .03 | .01 | .. | |
| | 25 000 | .... | .. | .03 | .01 | .. | |
| | 26 000 | .... | .. | .03 | .01 | .. | |
| | 27 000 | .... | .. | .03 | .01 | .. | |
| | 28 000 | .... | .. | .04 | .01 | .. | |
| | 29 000 | .... | .. | 05 | .01 | .. | |
| | 30 000 | 29 810 | .. | .08 | .. | .. | Failed. Extent of motion. Scale slightly disturbed in middle 14 in. north side. |
| | 10 900 | 10 830 | .. | .. | .. | .. | |
| | 10 800 | .... | .. | 1.06 | .. | .. | |
| | 200 | .... | .. | .09 | .02 | .. | Applying pressure on rockers here produces no change; piece springs back. |
| | 1 000 | .... | .. | .10 | .01 | .. | |
| | 2 000 | .... | .. | .10 | .01 | .. | |
| | 3 000 | .... | .. | .11 | ·01 | .. | |
| | 4 000 | .... | .. | .12 | .01 | .. | |
| | 5 000 | .... | .. | .13 | .01 | .. | |
| | 6 000 | .... | .. | .14 | .01 | .. | |
| | 7 000 | .... | .. | .15 | .01 | .. | |
| | 8 000 | .... | .. | .16 | .01 | .. | |
| | 9 000 | .... | .. | .21 | .. | .. | Going. |
| | 9 000 | .... | .. | .23 | .. | .. | |
| | 9 800 | 9 730 | .. | .39 | .. | .. | Going rapidly. Failed. |
| | 9 300 | .... | .. | .47 | .. | .. | |
| | 9 130 | .... | .. | .47 | .. | .. | |
| | 9 500 | .... | .. | .60 | .. | .. | |
| | 9 400 | .... | .. | .79 | .. | .. | |
| | 200 | .... | .. | .09 | .02 | .. | Specimen removed. |

Ordinate north at middle..... = .10 inch.
" west " ..... ..... = .03 "
Chord shortening ..... ..... = 00 "
Shortening on nort hside in middle 6 inches.. = .01 "
" " " 12 " = .01 "

| Description of Specimen | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In. | | |
| Mark, W 6· | 00 | 00 | .. | .09 | .03 | .. | |
| | 500 | .... | .. | .09 | .03 | .. | |
| Length, 55.93 in. | 5 000 | .... | .. | .12 | .01 | .. | |
| | 6 000 | .... | .. | .14 | .01 | .. | |
| Size, | 7 000 | .... | .. | .16 | .01 | .. | |
| 1.000 x 1.000 in. | 8 000 | .. | .. | .31 | .. | .. | Going. |
| | 8 000 | .... | .. | .38 | .. | .. | |
| Ordinate west | 8 000 | .... | .. | .38 | .. | .. | Later. |
| at 20 in. = .04 in. | 8 400 | 8 400 | .. | .54 | .. | .. | Failed. |
| | 8 000 | 8 000 | .. | .56 | .. | .. | |
| | 8 300 | .... | .. | .. | .. | .. | |
| | 8 100 | .... | .. | .72 | .. | .. | |
| | 500 | .... | .. | .11 | .. | .. | |
| | 500 | .... | .. | .08 | .01 | .. | Specimen placed .09 inch north of axis. |
| | 5 000 | .... | .. | .07 | .01 | .. | |
| | 6 000 | .... | .. | .07 | .01 | .. | |
| | 7 000 | .... | .. | .06 | .01 | .. | |
| | 8 000 | .... | .. | .03 | .01 | .. | |
| | 8 800 | 8 800 | .. | .. | .. | .. | Failed. |
| | 8 200 | .... | .21 | .. | .. | .. | |
| | 500 | .... | .. | .08 | .02 | .. | |
| | 500 | .... | .. | .08 | .01 | .. | Specimen placed .06 inch north of axis. |
| | 5 000 | .... | .. | .08 | .01 | .. | |
| | 6 000 | .... | .. | .08 | .01 | .. | |
| | 7 000 | .... | .. | .08 | .01 | .. | |
| | 8 000 | .... | .. | .08 | .01 | .. | |
| | 9 000 | .... | .. | .09 | .01 | .. | |
| | 10 000 | .... | .. | .10 | .01 | .. | |
| | 11 000 | .... | .. | .10 | .01 | .. | |
| | 12 000 | .... | .. | .11 | .01 | .. | |
| | 13 000 | .... | .. | .11 | .01 | .. | |
| | 14 000 | .... | .. | .12 | .01 | .. | |
| | 15 000 | .... | .. | .13 | .01 | .. | |
| | 16 000 | .... | .. | .14 | .01 | .. | |
| | 17 000 | .... | .. | .15 | .01 | .. | |
| | 18 000 | .... | .. | .16 | .01 | .. | |
| | 18 900 | 18 900 | .. | . | .. | .. | Failed. |
| | 8 700 | 8 700 | .. | .94 | .. | .. | Rocker free. |
| | 8 500 | .... | .. | .93 | .. | .. | |
| | 500 | .... | .. | .09 | .01 | .. | |
| | 500 | .... | .. | .09 | .01 | .. | Specimen placed .07 inch north of axis. |
| | 5 000 | ... | .. | .09 | .01 | .. | |
| | 10 000 | .... | .. | .10 | .01 | .. | |
| | 11 000 | .... | .. | .11 | .01 | .. | |
| | 12 000 | .... | .. | .11 | .01 | .. | |
| | 13 000 | .... | .. | .11 | .01 | .. | |
| | 14 000 | .... | .. | .12 | .01 | .. | |
| | 15 000 | .... | .. | .13 | .01 | .. | |
| | 16 000 | .... | .. | .13 | .01 | .. | |
| | 17 000 | .... | .. | .15 | .01 | .. | |
| | 18 000 | .... | .. | .16 | .01 | .. | |
| | 19 000 | .... | .. | .18 | .01 | .. | |
| | 20 000 | 20 000 | .. | .22 | .. | .. | Failed suddenly. Rocker free. |
| | 8 800 | 8 800 | .. | .99 | .. | .. | |
| | 7 700 | .... | .. | .79 | .. | .. | |
| | 7 200 | .... | .. | .59 | .. | .. | |
| | 6 800 | .... | .. | .39 | .. | .. | |
| | 6 400 | .... | .. | .29 | .. | .. | |
| | 5 900 | .... | .. | .24 | .. | .. | |
| | 5 000 | .... | .. | .18 | .. | .. | |
| | 4 000 | .... | .. | .15 | .. | .. | |
| | 3 000 | .... | .. | .14 | .. | .. | |
| | 2 000 | .... | .. | .11 | .. | .. | |
| | 1 000 | .... | .. | .10 | .. | .. | |
| | 500 | .... | .. | .09 | .01 | .. | Specimen removed. |

After Removing Specimen.

Ordinate north at middle = .09 inch.
" west " = .02 "
Chord shortening..... = 00 "
Scale nowhere disturbed.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In. | | |
| Mark, W 4. | 00 | 00 | .. | .06 | .04 | .. | |
| | 500 | .... | .. | .06 | .03 | .. | |
| Length, 60.01 in. | 1 000 | .... | .. | .06 | .03 | .. | |
| | 5 000 | .... | .. | .08 | .01 | .. | |
| Size, | 8 000 | 8 000 | .. | .. | .. | .. | Failed. |
| 1.000 x 1.000. | 5 000 | 5 000 | .. | .. | .. | .. | |
| | 500 | .... | .. | .06 | .. | .. | |
| | 5 000 | .... | .. | .10 | .. | .. | |
| | 6 000 | .... | .. | .10 | .02 | .. | |
| | 7 000 | .... | .. | .13 | .02 | .. | |
| | 8 000 | 8 000 | .. | .. | .. | .. | Failed. |
| | 7 200 | 7 200 | .. | .41 | .. | .. | |
| | 500 | .... | .. | .17 | .03 | .. | |
| | 500 | .... | .. | .05 | .. | .. | Applying pressure to rocker to bend bar and then removing pressure. |
| | 500 | .... | .. | .05 | .03 | .. | Specimen placed .06 inch north of axis. |
| | 5 000 | . | .. | .04 | .02 | .. | |
| | 6 000 | .... | .. | .04 | .01 | .. | |
| | 7 000 | .... | .. | .03 | .01 | .. | |
| | 8 000 | .... | .. | .03 | .01 | .. | |
| | 9 000 | .... | .. | .02 | .02 | .. | |
| | 10 000 | 10 000 | .. | .. | .. | .. | Failed. |
| | 7 300 | 7 300 | .47 | .. | .01 | .. | |
| | 500 | .... | .. | .04 | .03 | .. | |
| | 500 | .... | .. | .05 | .. | .. | Applying pressure to rocker to bend bar and then removing pressure. |
| | 500 | .... | .. | .05 | .04 | .. | Specimen placed .04 inch north of axis. |
| | 5 000 | .... | .. | .05 | .02 | .. | |
| | 6 000 | .... | .. | .05 | .02 | .. | |
| | 7 000 | .... | .. | .05 | .02 | .. | |
| | 8 000 | .... | .. | .05 | .02 | .. | |
| | 9 000 | .... | .. | .05 | .02 | .. | |
| | 10 000 | .... | .. | .05 | .02 | .. | |
| | 11 000 | .... | .. | .06 | .02 | .. | |
| | 12 000 | .... | . | .06 | .02 | .. | |
| | 13 000 | .... | .. | .07 | .02 | .. | |
| | 14 000 | .... | .. | .07 | .02 | .. | |
| | 15 000 | .... | .. | .08 | .02 | .. | |
| | 16 000 | .... | .. | .10 | .02 | .. | |
| | 17 000 | .... | .. | .11 | .02 | .. | |
| | 17 900 | 17 900 | .. | .. | .. | .. | Failed suddenly. Rocker free. |
| | 7 300 | 7 300 | .. | .97 | .. | .. | |
| | 500 | .... | . | .07 | .04 | .. | |
| | 500 | .... | .. | .05 | .. | .. | Applying pressure to rocker to bend bar and then removing pressure. Specimen removed. |

Ordinate north at middle = .06 inch.
" west " = .03 "

Scale not disturbed.

## CAMBRIA IRON COMPANY—TESTING LABORATORY.

JOHNSTOWN, PA., May, 1887.

### DETAIL REPORTS OF COMPRESSION TESTS OF 1-INCH SQUARE IRON, SERIES K, WITH HINGED-ENDS.

*Iron made by Union Iron Mills, Pittsburg, Pa.*

#### EXPLANATIONS.

All tested as from rolls; not cold straightened.

Ends of bars were plane and rested on hinged rocker-bearings with lubricated pins, the bearing surface of which is in plane of axis of the pins.

Deviations were taken at middle of bar, and are referred to the straight line passing through center of ends of bar. Deviations given for load 00 were measured before putting bar into testing machine by applying a straight-edge. Deviations under load were obtained by measuring with a steel scale the ordinates from fine fish cords, stretched parallel to bar by means of a clamp at bottom, to which one end of cord was fastened, and a similar clamp at top carrying grooved pulleys over which the cords passed, with weights attached to overhanging ends. Deviations under no load and under initial load of 200 or 500 pound are assumed to be equal.

The tests were made on a vertical 300 000-pound Emery Testing Machine.

Axis of pins was in all cases east and west.

All bars, except a few of the longest ones, were scribed every inch on all four sides, and the inches are referred to by number beginning at bottom.

Marked end of bar was down in all cases.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 4. | 00 | 00 | .. | 00 | .. | 00 | |
| | 200 | .... | .. | 00 | .. | 00 | |
| Length, 11.98 in. | 5 000 | .... | .. | 00 | .. | 00 | |
| | 10 000 | .... | .. | 00 | .. | 00 | |
| Size, | 20 000 | .... | .. | 00 | .. | 00 | |
| 1.005 x .995 in. | 30 000 | .... | .. | 00 | .. | 00 | |
| | 31 000 | .... | .. | 00 | .. | 00 | |
| Slight shear mark | 32 000 | .... | .. | 00 | .. | 00 | |
| at upper end. | 32 900 | 32 900 | .. | 00 | .. | 00 | Gone. Scaled at upper end. |
| | 32 500 | .... | .. | .005 | .. | 00 | |
| Ends square in | 31 800 | .... | .. | .04 | .. | 00 | |
| both directions. | 31 700 | .... | .. | .06 | .. | .. | |
| | | | | about | | | |
| | 31 500 | .... | .. | .07 | .. | .. | |
| | 32 400 | 32 400 | .. | .11 | .. | .. | Scaled all along. |
| | 32 000 | .... | .. | .13 | .. | .. | |
| | 200 | .... | .. | .15 | .. | 00 | |
| | 5 000 | .... | .. | .16 | .. | .. | |
| | 10 000 | .... | .. | .16 | .. | .. | |
| | 15 000 | .... | .. | .16 | .. | .. | |
| | 20 000 | .... | .. | .16 | .. | .. | |
| | 25 000 | .... | .. | .16 | .. | .. | |
| | 26 000 | .... | .. | .16 | .. | .. | |
| | 27 000 | .... | .. | .16 | .. | .. | |
| | 28 000 | .... | .. | .16 | .. | .. | |
| | 29 000 | .... | .. | .16 | .. | .. | |
| | 30 000 | .... | .. | .18 | .. | .. | Gone. |
| | 29 500 | .... | .. | .20 | .. | .. | |
| | 28 600 | .... | .. | .23 | .. | .. | Extent of rocking motion. Removed specimen. |

Chord shortening = .12 inch.
East and west perfectly straight.

Shortening Per Inch.

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| North Side | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| | .0 | .01 | .01 | .02 | .025 | .03 | .025 | .015 | .015 | .005 | .005 | .01 |
| South Side | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| | .01 | .01 | .005 | 0 | 0 | 0 | 0 | 0 | .005 | .015 | .015 | .02 |

Ordinates North.

| At | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | inches. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | .06 | .12 | .18 | .25 | .27 | .28 | .25 | .20 | .14 | .08 | .04 | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K—: | 00 | 00 | .01 | .. | .. | .015 | |
| | 200 | .... | .01 | .. | .. | .015 | |
| Length, 15.98 in. | 5 000 | .... | .01 | .. | .. | .015 | |
| | 10 000 | .... | .01 | .. | .. | .015 | |
| Size, | 15 000 | .... | .01 | .. | .. | .015 | |
| 1.000 x 1.000 in. | 20 000 | .... | .01 | .. | .. | .015 | |
| | 25 000 | .... | .01 | .. | .. | .015 | |
| | 26 000 | .... | .01 | .. | .. | .015 | |
| | 27 000 | .... | .01 | .. | .. | .015 | |
| | 28 000 | .... | .01 | .. | .. | .015 | |
| | 29 000 | .... | .01 | .. | .. | .... | |
| | 30 000 | .... | .01 | .. | .. | .015 | |
| | 31 000 | .... | .01 | .. | .. | .... | |
| | 32 000 | .... | .01 | .. | .. | .... | |
| | 33 000 | .... | .01 | .. | .. | .025 | |
| | 34 000 | .... | .01 | .. | .. | .025 | |
| | 34 300 | 34 300 | .01 | .. | .. | .065 | Gone. |
| | 32 000 | 32 000 | .03 | .. | .. | .075 | |
| | 31 400 | .... | .03 | .. | .. | .095 | |
| | 31 000 | .... | .03 | .. | .. | .155 | |
| | 30 900 | .... | .03 | .. | .. | .295 | |
| | 200 | .... | .02 | .. | .. | .305 | |
| | 5 000 | .... | .02 | .. | .. | .325 | |
| | 10 000 | .... | .02 | .. | .. | .325 | |
| | 15 000 | .... | .02 | .. | .. | .335 | |
| | 20 000 | .... | .01 | .. | .. | .335 | |
| | 25 000 | .... | .01 | .. | .. | .335 | Removed specimen. |

Ordinate east at middle.. = .28 inch.
" south at middle. = .01 "
Chord shortening..... = .13 "

Shortening Per Inch.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| East Side | 1 to 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 to 16 | |
| | 0 | .01 | .01 | .025 | .025 | .02 | .01 | .01 | 0 | |
| West Side | 1 | 2 | 3 | 4 | 5 | 6 to 12 | 13 | 14 | 15 | 16 |
| | .02 | .02 | .015 | .005 | .005 | 0 | .01 | .01. | .02 | .02 |

| Description of Specimen | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 1 | 00 | 00 | .. | 00 | .. | .02 | |
| | 2 000 | .... | .. | 00 | .. | .02 | |
| Length, 19.99 in. | 5 000 | .... | .. | .01 | .. | .02 | |
| | 10 000 | .... | .. | .01 | .. | .02 | |
| Size, | 15 000 | .... | .. | .01 | .. | .02 | |
| 1.003 x 1.000 in. | 20 000 | .... | .. | .01 | .. | .02 | |
| | 25 000 | .... | .. | .01 | .. | .02 | |
| | 26 000 | .... | .. | .01 | .. | .02 | |
| | 27 000 | .... | .. | .01 | .. | | |
| | 28 000 | .... | .. | .01 | .. | | |
| | 29 000 | .... | .. | .01 | .. | | |
| | 30 000 | .... | .. | .01 | .. | .02 | |
| | 31 000 | .... | .. | .01 | .. | | |
| | 32 000 | .... | .. | .01 | .. | .02 | |
| | 33 000 | .... | .. | .015 | .. | .03 | |
| | 33 500 | .... | .. | .015 | .. | .03 | |
| | 33 600 | .... | .. | .02 | .. | | |
| | 34 000 | .... | .. | .02 | .. | .04 | |
| | 35 000 | 34 910 | .. | .04 | .. | .10 | Fails. |
| | 30 000 | .... | .. | .05 | .. | .17 | |
| | 29 000 | .... | .. | .05 | .. | .19 | |
| | 28 600 | .... | .. | .05 | .. | .21 | |
| | 28 000 | .... | .. | .07 | .. | .28 | |
| | 27 000 | .... | .. | .09 | .. | .38 | |
| | 26 000 | .... | .. | .13 | .. | .45 | |
| | 25 000 | .... | .. | .16 | .. | .50 | |
| | 200 | .... | .. | .12 | .. | .44 | |
| | 2 000 | .... | .. | .12 | .. | .45 | |
| | 5 000 | .... | .. | .12 | .. | .45 | |
| | 10 000 | .... | .. | .12 | .. | .46 | |
| | 15 000 | .... | .. | .14 | .. | .47 | |
| | 20 000 | .... | .. | .15 | .. | .49 | Removed specimen. |

Ordinate east at middle.. = .42 inch.
" north at middle = .10 "
Chord shortening..... = .10 "

Shortening Per Inch.

| East Side | 1 to 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 to 20 |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | .01 | .01 | .02 | .02 | .02 | .02 | .01 | 0 |

| West Side | 1 | 2 | 3 | 4 to 8 | 9 | 10 | 11 | 12 | 13 to 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | .02 | .02 | .01 | 0 | —.005 | —.005 | —.005 | —.005 | 0 | —.01 | —.01 | —.02 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | .. | In. | |
| Mark, K 1: | 00 | .... | .. | 00 | .. | 00 | |
| | 200 | .... | .. | 00 | .. | 00 | |
| Length, 20.00 in. | 5 000 | ... | .. | .01 | .. | 00 | |
| | 10 000 | .... | .. | .01 | .. | 00 | |
| Size, | 15 000 | .... | .. | .01 | .. | 00 | |
| 1.003 x 1.000 in. | 20 000 | .... | .. | .01 | .. | 00 | |
| | 25 000 | .... | .. | .01 | .. | 00 | |
| East and west | 26 000 | .... | .. | .01 | .. | 00 | |
| straight. | 27 000 | .... | .. | .01 | .. | .. | |
| | 28 000 | .... | .. | .01 | .. | .. | |
| North and south | 29 000 | .... | .. | .01 | .. | 00 | |
| straight. | 30 000 | .... | .. | .01 | .. | 00 | |
| | 31 000 | .... | .. | .01 | .. | 00 | |
| | 32 000 | .... | .. | 01 | .. | 00 | |
| | 33 000 | .... | .. | .02 | .. | 00 | |
| | 34 000 | 33 910 | .. | .03 | .. | .. | Going. Fails. |
| | 31 500 | .... | .. | .08 | .. | .01 | |
| | 20 000 | .... | .. | .35 | .. | .02 | |
| | 200 | .... | .. | .28 | .. | .01 | |
| | 5 000 | .... | .. | .30 | .. | .01 | |
| | 10 000 | .... | .. | .32 | .. | .01 | |
| | 15 000 | .... | .. | .35 | .. | .01 | |
| | 16 000 | .... | .. | .36 | .. | .. | |
| | 17 000 | .... | .. | .37 | .. | .. | |
| | 18 000 | .... | .. | .38 | .. | .. | |
| | 19 000 | .... | .. | .40 | .. | .. | Gone. Removed. |

Ordinate north at middle = .33 inch.
" east " = .01 "
Chord shortening ..... = .04 "

Shortening Per Inch.

| North Side | 1 to 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 to 20 |
|---|---|---|---|---|---|---|---|---|
| | 0 | 0 | .01 | .02 | .02 | .01 | .01 | 00 |

| South Side | 1 | 2 to 9 | 10 | 11 | 12 | 13 | 14 to 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|
| | .01 | 0 | —.005 | —.005 | —.005 | —.005 | 0 | .01 | .01 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| Mark, K 1· | 00 | .... | .025 | .. | 00 | .. | |
| | 200 | .... | .025 | .. | 00 | .. | |
| Length, 22.00 in. | 5 000 | .... | .025 | .. | .01 | .. | |
| | 10 000 | .... | .025 | .. | .01 | .. | |
| Size, | 15 000 | .... | .015 | .. | .01 | .. | |
| 1.003 × 1.000 in. | 20 000 | .... | .015 | .. | .01 | .. | |
| | 25 000 | .... | .015 | .. | .01 | .. | |
| Ordinate south, | 26 000 | .... | .015 | .. | .01 | .. | |
| = .025. | 27 000 | .... | .025 | .. | .01 | .. | |
| | 28 000 | .... | .025 | .. | .01 | .. | |
| East and west | 29 000 | .... | .025 | .. | .. | .. | |
| straight. | 30 000 | .... | .025 | .. | .01 | .. | |
| | 31 000 | .... | .. | .. | .. | .. | |
| | 32 000 | .... | .. | .. | .. | .. | |
| | 33 000 | .... | .. | .. | .. | .. | |
| | 34 000 | 33 910 | .035 | .. | .01 | .. | Going. |
| | 20 000 | 19 940 | .355 | .. | .. | .. | |
| | 19 000 | .... | .415 | .. | .. | .. | |
| | 18 000 | .... | .475 | .. | .. | .. | |
| | 200 | .... | .365 | .. | 00 | .. | |
| | 5 000 | .... | .395 | .. | 00 | .. | |
| | 10 000 | .... | .425 | .. | 00 | .. | |
| | 15 000 | .... | .475 | .. | .. | .01 | Removed specimen. |

Ordinate south at middle, = .38 inch.
" west " = .02 "
Chord shortening..... = .04 "

Shortening Per Inch.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| South Side | 1 to 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 to 22 |
| | 00 | .01 | .01 | .01 | .015 | .015 | .015 | .01 | 0 |
| North Side. | 1 to 10 | 11 | 12 | 13 to 20 | | | 21 | 22 | |
| | 0 | —.005 | —.005 | 0 | | | .01 | .02 | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 11· | 00 | 00 | .. | .04 | .. | .02 | |
| | 200 | .... | .. | .04 | .. | .02 | |
| Length, 24.00 in. | 5 000 | .... | .. | .04 | .. | .02 | |
| | 10 000 | .... | .. | .04 | .. | .02 | |
| Size, | 15 000 | .... | .. | .05 | .. | .02 | |
| 1.006 × 1.003 in. | 20 000 | .... | .. | .05 | .. | .02 | |
| | 25 000 | .... | .. | .05 | .. | .02 | |
| Ordinates | 26 000 | .... | .. | .05 | .. | .. | |
| East = .02 | 27 000 | .... | .. | .06 | .. | .. | |
| North = .04 | 28 000 | .... | .. | .06 | .. | .. | |
| | 29 000 | .... | .. | .06 | .. | .. | |
| | 30 000 | .... | .. | .06 | .. | .02 | |
| | 31 000 | .... | .. | .07 | .. | .. | |
| | 31 500 | 31 230 | .. | .09 | .. | .. | Gone. |
| | 17 000 | 16 850 | .. | .40 | .. | .. | |
| | 16 000 | .... | .. | .45 | .. | .. | |
| | 16 000 | .... | .. | .55 | .. | .06 | |
| | 200 | .... | .. | .42 | .. | .06 | |
| | 5 000 | .... | .. | .46 | .. | .06 | |
| | 10 000 | .... | .. | .52 | .. | .06 | Limit of rocking motion. Removed specimen. |

Ordinate north at middle, = .48 inch.
" east " = .05 "
Chord shortening, ..... = .04 "

Shortening Per Inch.

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| North Side. | 1 to 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 to 24 |
| | 0 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | .005 | 0 |
| South Side. | 1 | 2 to 23 | 24 | | | | | | | | |
| | .02 | 0 | .02 | | | | | | | | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 2· | 00 | 00 | .. | .02 | .. | .02 | |
| Ordinates. | 200 | .... | .. | .02 | .. | .02 | |
| East, = .02 in. | 5 000 | .... | .. | .02 | .. | .02 | |
| North, = .02 in. | 10 000 | .... | .. | .02 | .. | .02 | |
| | 15 000 | .... | .. | .02 | .. | .02 | |
| | 20 000 | .... | .. | .02 | .. | .02 | |
| Length, 26.02 in. | 25 000 | .... | .. | .03 | .. | .02 | |
| | 26 000 | .... | .. | .03 | .. | .. | |
| Size. | 27 000 | .... | .. | .03 | .. | .. | |
| 1.008 × 1.000 in. | 28 000 | .... | .. | .03 | .. | .. | |
| | 29 000 | .... | .. | .03 | .. | .. | |
| | 30 000 | .... | .. | .03 | .. | .02 | |
| | 31 000 | .... | .. | .03 | .. | .. | |
| | 32 000 | .... | .. | .03 | .. | .. | |
| | 33 000 | .... | .. | .04 | .. | .02 | Scaling. |
| | 33 900 | 33 750 | .. | .05 | .. | .. | Fails. |
| | 14 700 | 14 590 | .. | .56 | .. | .. | |
| | 200 | .... | .. | .38 | .. | .05 | |
| | 5 000 | .... | .. | .44 | .. | .05 | |
| | 8 000 | .... | .. | .48 | .. | .. | |
| | 10 000 | .... | .. | .51 | .. | .. | |
| | 11 000 | .... | .. | .52 | .. | .. | |
| | 12 000 | .... | .. | .54 | .. | .. | |
| | 13 000 | .... | .. | .56 | .. | .. | Probably at limit of rocking motion. |
| | 14 000 | .... | .. | .58 | .. | .. | |
| | 15 000 | .... | .. | .62 | .. | .. | Scales rapidly. Piece removed. |

Ordinate north at middle, = .52 inch.
" east " = .06 "
Chord shortening,..... = .04 "

Shortening Per Inch.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| North Side. | 1 to 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 to 26 |
| | 0 | .01 | .01 | .015 | .015 | .015 | .015 | .01 | 0 |
| South Side. | 1 to 11 | 12 | 13 | 14 | 15 to 26 | | | | |
| | 0 | —.005 | —.005 | —.005 | 0 | | | | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | | |
| Mark, K 3· | 00 | 00 | .. | .02 | .05 | .. | |
| | 200 | 200 | .. | .02 | .05 | .. | |
| Length, 28.01 in. | 5 000 | .... | .. | .02 | .04 | .. | |
| | 10 000 | .... | .. | .02 | .05 | .. | |
| Size, | 15 000 | .... | .. | .03 | .05 | .. | |
| 1.000 x 1.000 in. | 20 000 | .... | .. | .03 | .05 | .. | |
| | 25 000 | .... | .. | .03 | .04 | .. | |
| | 26 000 | .... | .. | .03 | .04 | .. | |
| | 27 000 | .... | .. | .03 | .05 | .. | |
| | 28 000 | .... | .. | .03 | .05 | .. | |
| | 29 000 | .... | .. | .03 | .05 | .. | |
| | 30 000 | .... | .. | .03 | .07 | .. | |
| | 31 000 | .... | .. | .03 | .07 | .. | |
| | 32 000 | 32 000 | .. | .08 | .09 | .. | Scaling. Gone. Bent east first, then south. |
| | 14 000 | 14 000 | .. | .61 | .25 | .. | |
| | 200 | .... | .. | .41 | .23 | .. | |
| | 5 000 | .... | .. | .46 | .23 | .. | |
| | 10 000 | .... | .. | .55 | .23 | .. | |
| | 11 000 | .... | .. | .58 | .23 | .. | |
| | 12 000 | .... | .. | .61 | .23 | .. | |
| | 13 000 | .... | .. | .66 | .23 | .. | Extent of rocking motion. Removed specimen. |

Ordinate north at middle ..... = .50 inch.
" west " " ..... = .27 "
Chord shortening ..... = .05 "
Maximum shortening at center west side = .015 "

Shortening Per Inch.

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| North Side | 1 to 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 to 28 |
| | 0 | .01 | .01 | .015 | .015 | .015 | .015 | .01 | .01 | .005 | 0 |
| South Side | 1 | 2 to 11 | 12 | 13 to 27 | 28 | | | | | | |
| | .01 | 0 | 0 | 0 | .01 | | | | | | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 3: | 00 | 00 | .06 | .. | .. | .04 | |
| | 200 | .... | .06 | .. | .. | .04 | |
| Length, 28.02 in. | 5 000 | .... | .06 | .. | .. | .05 | |
| | 10 000 | .... | .06 | .. | .. | .05 | |
| Size, | 15 000 | .... | .07 | .. | .. | .05 | |
| 1.000 x 1.000 in. | 20 000 | .... | .08 | .. | .. | .05 | |
| | 23 000 | .... | .08 | .. | .. | .. | |
| Twist, .04 in. left. | 24 000 | .... | .08 | .. | .. | .. | |
| | 25 000 | .... | .08 | .. | .. | .06 | |
| | 26 000 | .... | .08 | .. | .. | .06 | |
| | 27 000 | ... | .08 | .. | .. | .06 | |
| | 28 000 | .... | .08 | .. | .. | .06 | |
| | 29 000 | .... | .09 | .. | .. | .06 | |
| | 30 000 | .... | .095 | .. | .. | .07 | |
| | 30 700 | 30 700 | .. | .. | .. | .. | Scaled on east side first. Fails. |
| | 13 900 | 13 900 | .63 | .. | .. | .13 | |
| | 200 | .... | .42 | .. | .. | .10 | |
| | 5 000 | .... | .46 | .. | .. | .11 | |
| | 6 000 | .... | .48 | .. | .. | .11 | |
| | 7 000 | .... | .49 | .. | .. | .11 | |
| | 8 000 | .... | .52 | .. | .. | .12 | |
| | 9 000 | .... | .53 | .. | .. | .12 | |
| | 10 000 | .... | .55 | .. | .. | .12 | |
| | 11 000 | .... | .58 | .. | .. | .12 | |
| | 12 000 | .... | .60 | .. | .. | .13 | |
| | 13 000 | .... | .62 | .. | .. | .13 | |
| | 13 400 | .... | .66 | .. | .. | .13 | Extent of rocking motion. Removed specimen. |

Ordinate south at middle = .49 inch.
" east " = .10 "
Chord shortening..... = .03 "

Shortening Per Inch.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| South Side | 1 to 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 to 28 |
| | 0 | .005 | .01 | .015 | .015 | .01 | .01 | .01 | .005 | 0 |
| North Side | 1 to 12 | 13 | 14 | 15 | 16 | | | | | |
| | 0 | —.005 | —.005 | —.005 | —.005 | | | | | |

| Description of Specimen. | Load Lbs. | Load. Lbs per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 5· | 00 | 00 | .. | .06 | .. | 00 | |
| | 200 | .... | .. | .06 | .. | .00 | |
| Length, 30.04 in. | 5 000 | ... | .. | .07 | .. | 00 | |
| | 10 000 | .... | .. | .07 | .. | 00 | |
| Size, | 15 000 | .... | .. | .08 | .. | 00 | |
| 1.010 x 1.015 in. | 20 000 | .... | .. | .09 | .. | 00 | |
| | 25 000 | .... | .. | .10 | .. | 00 | |
| .06 in. twist left. | 26 000 | .... | .. | .10 | .. | 00 | |
| | 27 000 | .... | .. | .10 | .. | .. | |
| | 28 000 | .... | .. | 10 | .. | .. | |
| | 29 000 | .... | .. | .11 | .. | .. | |
| | 30 000 | 29 270 | .. | .12 | .. | .. | Fails. |
| | 13 000 | 12 780 | .. | .65 | .. | .02 | |
| | 200 | .... | .. | .44 | .. | .03 | |
| | 5 000 | .... | .. | .49 | .. | .. | |
| | 6 000 | .... | .. | .52 | .. | .. | |
| | 7 000 | .... | .. | .54 | .. | .. | |
| | 8 000 | .... | .. | .56 | .. | .. | |
| | 9 000 | .... | .. | .59 | .. | .. | |
| | 10 000 | .... | .. | .62 | .. | .. | |
| | 11 000 | .... | .. | .66 | .. | .. | |
| | 12 000 | 11 710 | .. | .76 | .. | .. | Maximum and extent of rocking motion. Specimen removed. |

Ordinate north at middle = .56 inch.
" east " = .03 "
Chord shortening ..... = .05 "

Shortening Per Inch.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| North Side | 1 to 13 | 14 | 15 | 16 | 17 | 18 | 19 to 30 |
| | 0 | .01 | .01 | .01 | .01 | .01 | 0 |
| South Side | 1 to 12 | 13 | 14 | 15 | 16 | 17 | 18 to 30 |
| | 0 | —.005 | —.005 | —.005 | —.005 | —.005 | 0 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 18· | 00 | 00 | .03 | .. | .. | .02 | |
| | 200 | .... | .03 | .. | .. | .02 | |
| Length, 30.03 in. | 5 000 | .... | .02 | .. | .. | .02 | |
| | 10 000 | .... | .02 | .. | .. | .02 | |
| Size, | 15 000 | .... | .03 | .. | .. | .02 | |
| 1.005 x .995 in. | 20 000 | .... | .03 | .. | .. | .02 | |
| | 25 000 | .... | .03 | .. | .. | .02 | |
| | 26 000 | .... | .03 | .. | .. | .02 | |
| | 27 000 | .... | .03 | .. | .. | .02 | |
| | 28 000 | .... | .03 | .. | .. | .. | |
| | 29 000 | .... | .04 | .. | .. | .02 | |
| | 30 000 | .... | .04 | .. | .. | .02 | |
| | 31 000 | .... | .04 | .. | .. | .02 | |
| | 32 000 | .... | .04 | .. | .. | .01 | |
| | 33 000 | 33 000 | .05 | .. | .. | .. | Going. Fails. |
| | 12 000 | 12 000 | .73 | .. | .. | .02 | |
| | 200 | .... | .50 | .. | .. | .03 | |
| | 5 000 | .... | .58 | .. | .. | .02 | |
| | 6 000 | .... | .60 | .. | .. | .. | |
| | 7 000 | .... | .63 | .. | .. | .. | |
| | 8 000 | .... | .66 | .. | .. | .. | |
| | 9 000 | .... | .69 | .. | .. | .. | |
| | 10 000 | .... | .72 | .. | .. | .. | |
| | 11 000 | .... | .73 | .. | .. | .. | Before 11 000, upper rocker had reached extent of motion. Removed specimen |

Ordinate south at middle = .51 inch.
" west " = .01 "
Chord shortening ..... = .03 "

Shortening Per Inch.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| South Side | 1 to 13 | 14 | 15 | 16 | 17 | 18 | 19 to 30 |
| | 0 | .01 | .01 | .01 | .01 | .01 | 0 |
| North Side | 1 to 13 | 14 | 15 | 16 | 17 | 18 to 30 | |
| | 0 | —.005 | —.005 | —.005 | —.005 | 0 | |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In. | | |
| Mark, K 3: | 00 | 00 | .. | 00 | .02 | .. | |
| | 200 | .... | .. | 00 | .02 | .. | |
| Length, 32.00 in. | 5 000 | .... | .. | 00 | .02 | .. | |
| | 10 000 | .... | .. | .005 | .02 | . | |
| Size, | 15 000 | .... | .. | .005 | .02 | .. | |
| 1.000 x 1.000 in. | 20 000 | .... | .. | 00 | .02 | .. | |
| | 25 000 | .... | .. | 00 | .03 | .. | |
| Right hand twist .10 in. on corners .06 in. at centers of sides. | 26 000 | .... | .01 | .. | .03 | .. | |
| | 27 000 | .... | 00 | 00 | .03 | .. | |
| | 28 000 | .... | .. | 00 | .03 | .. | |
| | 29 000 | .... | .. | 00 | .03 | .. | |
| | 30 000 | .... | .. | 00 | .035 | .. | |
| Defect in bar from 8 to 12 in. on west side. | 31 000 | .... | .. | 00 | .035 | .. | |
| | 32 000 | .... | .. | 00 | .04 | .. | |
| | 33 000 | .... | .. | 00 | .04 | .. | |
| | 34 000 | .... | .. | 00 | .04 | .. | |
| | 34 900 | 34 900 | .. | .05 | .. | .. | Fails.—These measurements at middle, but maximum deviation is at 13 in. |
| | 24 900 | 24 900 | .. | .. | .32 | .. | |
| | 13 000 | .... | .. | .50 | .39 | .. | |
| | 200 | .... | .. | .26 | .31 | .. | |
| | 200 | .... | .. | .29 | .34 | .. | These taken at 13 in. as are following measurements. |
| | 5 000 | .... | .. | .32 | .34 | .. | |
| | 6 000 | .... | .. | .33 | .35 | . | |
| | 7 000 | .... | .. | .34 | .36 | .. | |
| | 8 000 | .... | .. | .36 | .36 | .. | |
| | 9 000 | ... | .. | .38 | .36 | .. | |
| | 10 000 | .... | .. | .41 | .38 | .. | |
| | 11 000 | .... | .. | .45 | .38 | .. | |
| | 12 000 | .... | .. | .47 | .38 | .. | |
| | 13 000 | .... | .. | .52 | .41 | .. | |
| | 13 500 | 13 500 | .. | .58 | .41 | .. | Gone. Specimen removed. |

Ordinate north, maximum at 13 inch = .35 inch.
" east " " = .31 "
Chord shortening..... = .05 "

Shortening Per Inch.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| North Side | 1 to 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 to 32 |
| | 0 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | 0 |

South and East Sides { Lengthening on south side is not perceptible. Maximum shortening on east side is .01 in. per inch.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| Mark, K 4· | 00 | 00 | .. | 00 | .. | .04 | |
| | 200 | .... | .. | 00 | .. | .01 | |
| Length, 33.95 in. | 5 000 | .... | .. | 00 | .. | .03 | |
| | 10 000 | .... | .. | 00 | .. | .03 | |
| No twist. | 15 000 | .... | .. | .01 | .. | .03 | |
| | 20 000 | .... | .. | .01 | .. | .04 | |
| Size. | 25 000 | .... | .. | .02 | .. | .04 | |
| 1.005 x .997 in. | 26 000 | .... | .. | .02 | .. | .04 | |
| | 27 000 | .... | .. | .02 | .. | .04 | |
| | 28 000 | .... | .. | .02 | .. | .04 | |
| | 29 000 | .... | .. | .02 | .. | .04 | |
| | 30 000 | .... | .. | .03 | .. | .05 | |
| | 31 000 | .... | .. | .04 | .. | .07 | |
| | 31 000 | 30 940 | .. | .. | .. | .. | Gone. |
| | 10 000 | 9 980 | .. | .76 | .. | .. | |
| | 200 | .... | .. | .44 | .. | .09 | |
| | 5 000 | .... | .. | .54 | .. | .09 | |
| | 6 000 | .... | .. | .58 | .. | .. | |
| | 7 000 | .... | .. | .61 | .. | .. | |
| | 8 000 | .... | .. | .65 | .. | .. | |
| | 9 000 | .... | .. | .70 | .. | .. | |
| | 9 800 | 9 780 | .. | .76 | .. | .. | Extent of rocking motion. Removed specimen. |

Ordinate north at middle = .44 inch.
" east " = .07 "
Chord shortening..... = .02 "

Shortening Per Inch.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| North Side | 1 to 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 to 34 |
| | 0 | .005 | .005 | .01 | .01 | .01 | .01 | .005 | .005 | 0 |

| | | | | | | |
|---|---|---|---|---|---|---|
| South Side | 1 to 15 | 16 | 17 | 18 | 19 | 20 to 34 |
| | 0 | —.005 | —.005 | —.005 | —.005 | 00 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In | In. | | |
| Mark, K 4: | 00 | 00 | .. | .05 | .04 | .. | |
| | 200 | .... | .. | .05 | .04 | .. | |
| Length, 33.96 in. | 5 000 | .... | .. | .05 | .03 | .. | |
| | 10 000 | .... | .. | .06 | .03 | .. | |
| Size, | 15 000 | .... | .. | .07 | .03 | .. | |
| 1.005 x .997 in. | 20 000 | .... | .. | .08 | .03 | .. | |
| | 21 000 | .... | .. | .09 | .03 | .. | |
| Twist, .03 in. | 22 000 | .... | .. | .09 | .03 | .. | |
| | 23 000 | .... | .. | .09 | .03 | .. | |
| | 24 000 | .... | .. | .10 | .03 | .. | |
| | 25 000 | .... | .. | .11 | .03 | .. | |
| | 26 000 | 26 000 | .. | .13 | .03 | .. | Going slowly. |
| | 12 200 | 12 200 | .. | .57 | .. | .. | Went very slowly. Scale slightly disturbed at middle of north side. |
| | 200 | .... | .. | .29 | .02 | .. | |
| | 1 000 | .... | .. | .30 | .02 | .. | |
| | 2 000 | .... | .. | .31 | .. | .. | |
| | 3 000 | .... | .. | .32 | .. | .. | |
| | 4 000 | .... | .. | .33 | .. | .. | |
| | 5 000 | .... | .. | .34 | .. | .. | |
| | 6 000 | .... | .. | .36 | .. | .. | |
| | 7 000 | .... | .. | .39 | .. | .. | |
| | 8 000 | .... | .. | .41 | .. | .. | |
| | 9 000 | .... | .. | .43 | .. | .. | |
| | 10 000 | .... | .. | .48 | .. | .. | |
| | 11 000 | .... | .. | .52 | .. | .. | |
| | 12 000 | .... | .. | .57 | .. | .. | |
| | 12 500 | .... | .. | .63 | .. | .. | |
| | 10 900 | .... | .. | .73 | .. | .. | |
| | 200 | .... | .. | .41 | .. | .. | Specimen removed. |

Ordinate north at middle..... = .40 inch.
" east and west at middle = 00 "
Chord shortening..... = .03 "
The middle 8 inches on north side shortened altogether .03 inch.
" 8 " south " lengthened " .02 "

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | In. | |
| Mark, K 4· | 00 | 00 | .. | .06 | .. | .03 | |
| | 200 | .... | .. | .06 | .. | .03 | |
| Length, 36 04 in. | 5 000 | .... | .. | .07 | .. | .01 | |
| | 10 000 | .... | .. | .08 | .. | 00 | |
| Left-hand twist | 15 000 | .... | .. | .10 | .01 | .. | |
| = .03 in. | 16 000 | .... | .. | .11 | .01 | .. | |
| | 17 000 | .... | .. | .11 | .01 | .. | |
| Size, | 18 000 | .... | .. | .12 | .01 | .. | |
| 1.010 x .995 in. | 19 000 | .... | .. | .12 | .01 | .. | |
| | 20 000 | .... | .. | .13 | .01 | .. | |
| | 21 000 | 20 900 | .. | .. | .. | .. | Gone. Scales slightly on north side. |
| | 14 000 | 13 930 | .. | .53 | .. | .. | |
| | 200 | .... | .. | .20 | .. | .01 | |
| | 5 000 | .... | .. | .25 | .01 | .. | |
| | 6 000 | .... | .. | .26 | .01 | .. | |
| | 7 000 | .... | .. | .26 | .01 | .. | |
| | 8 000 | .... | .. | .29 | .01 | .. | |
| | 9 000 | .... | .. | .32 | .01 | .. | |
| | 10 000 | .... | .. | .33 | 00 | .. | |
| | 11 000 | .... | .. | .38 | 00 | .. | |
| | 12 000 | .... | .. | .43 | 00 | .. | |
| | 12 980 | 12 920 | .. | .56 | 00 | .. | Removed specimen. |

Ordinate north at middle = .29 inch.
" east " = 00 "
Chord shortening..... = .01 "

Shortening Per Inch.

| North Side | 16 | 17 | 18 | 19 | 20 | 21 |
|---|---|---|---|---|---|---|
| | .01 | .01 | .01 | .01 | .01 | .01 |

South Side uncertain.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 4: | 00 | 00 | .03 | .. | .. | .04 | |
| | 200 | .... | .03 | .. | .. | .04 | |
| Length, 36.04 in. | 5 000 | .... | .04 | .. | .. | .04 | |
| | 10 000 | .... | .05 | .. | .. | .04 | |
| Size, | 15 000 | .... | .05 | .. | .. | .04 | |
| 1.010 x .995 in. | 20 000 | 19 900 | .. | .. | .. | .. | Gone. Went rather slowly. |
| | 13 000 | 12 930 | .. | .. | .. | .. | |
| | 11 200 | 11 140 | .64 | .. | .. | .03 | |
| | 200 | .... | .30 | .. | .. | .03 | |
| | 1 000 | .... | .30 | .. | .. | .. | |
| | 2 000 | .... | .32 | .. | .. | .. | |
| | 3 000 | .... | .33 | .. | .. | .. | |
| | 4 000 | .... | .35 | .. | .. | .. | |
| | 5 000 | .... | .37 | .. | .. | .. | |
| | 6 000 | .... | .40 | .. | .. | .. | |
| | 7 000 | .... | .44 | .. | .. | .. | |
| | 8 000 | .... | .47 | .. | .. | .. | |
| | 9 000 | .... | .52 | .. | .. | .. | |
| | 10 000 | .... | .57 | .. | .. | .. | |
| | 11 000 | .... | .66 | .. | .. | .. | |
| | 11 300 | 11 240 | .. | .. | .. | .. | Gone. |
| | 10 300 | .... | .83 | .. | .. | .. | |
| | 10 200 | .... | .88 | .. | .. | .. | |
| | 200 | .... | .51 | .. | .. | .. | Removed specimen. |

Ordinate south at middle = .52 inch.
" east " = .04 "
Chord shortening..... = .02 "

Shortening Per Inch.

| South Side | 1 to 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 to 36 |
|---|---|---|---|---|---|---|---|---|
| | 0 | .0066 | .0066 | .0066 | .0066 | .0066 | .0066 | 0 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | | |
| Mark K, 5: | 00 | 00 | .. | .02 | .02 | .. | |
| | 200 | .... | .. | .02 | .02 | .. | |
| Length, 38.00 in. | 5 000 | .... | .. | .02 | .01 | .. | |
| | 10 000 | .... | .. | .03 | .01 | .. | |
| Size, | 15 000 | .... | .. | .03 | .01 | .. | |
| 1.010 x 1.010 in. | 20 000 | .... | .. | .04 | .01 | .. | |
| | 21 000 | .... | .. | .04 | .. | .. | |
| | 22 000 | .... | .. | .05 | .01 | .. | |
| | 23 000 | .... | .. | .05 | .02 | .. | |
| | 24 000 | 23 320 | .. | .07 | .. | .. | Gone suddenly. |
| | 11 500 | 11 220 | .. | .60 | .. | .. | |
| | 200 | .... | .. | .24 | .02 | .. | |
| | 1 000 | .... | .. | .25 | .. | .. | |
| | 2 000 | .... | .. | .25 | .. | .. | |
| | 3 000 | .... | .. | .27 | .. | .. | |
| | 4 000 | .... | .. | .29 | .. | .. | |
| | 5 000 | .... | .. | .31 | .. | .. | |
| | 6 000 | .... | .. | .33 | .. | .. | |
| | 7 000 | .... | .. | .35 | .. | .. | |
| | 8 000 | .... | .. | .39 | .. | .. | |
| | 9 000 | .... | .. | .43 | .. | .. | |
| | 10 000 | .... | .. | .49 | .. | .. | |
| | 11 000 | .... | .. | .56 | .. | .. | |
| | 11 800 | 11 510 | .. | .67 | .. | .. | |
| | 10 200 | .... | .. | .75 | .. | .. | |
| | 200 | .... | .. | .27 | .01 | .. | Removed specimen. |

Ordinate north at middle = .37 inch.
" west " = .03 "
Chord shortening..... = .01 "

Shortening Per Inch.

| | | | | | | |
|---|---|---|---|---|---|---|
| North Side | 1 to 17 | 18 | 19 | 20 | 21 | 20 to 38 |
| | 0 | .0067 | .0067 | .0067 | .0067 | 0 |

| Description of Specimen. | Load Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 5· | 00 | 00 | 00 | .. | .. | .03 | |
| | 200 | .... | 00 | .. | .. | .03 | |
| Length, 38.02 in. | 5 000 | .... | 00 | .. | .. | .04 | |
| | 10 000 | .... | 00 | .. | .. | .05 | |
| Size, | 15 000 | .... | 00 | .. | .. | .05 | |
| 1.010 x 1.015 in. | 16 000 | .... | 00 | .. | .. | .05 | |
| | 17 000 | .... | 00 | .. | .. | .05 | |
| Fairly straight. | 18 000 | .... | 00 | .. | .. | .06 | |
| | 19 000 | .... | .005 | .. | .. | .06 | |
| No twist. | 20 000 | .... | .005 | .. | .. | .06 | |
| | 21 000 | .... | .005 | .. | .. | 06 | |
| | 22 000 | .... | .01 | .. | .. | .06 | |
| | 23 000 | .... | .01 | .. | .. | .06 | |
| | 24 000 | .... | .01 | .. | .. | 06 | |
| | 25 000 | .... | .01 | .. | .. | .06 | |
| | 26 000 | .... | .01 | .. | .. | .07 | |
| | 27 000 | .... | .02 | .. | .. | .07 | |
| | 28 000 | .... | .02 | .. | .. | .08 | |
| | 29 000 | .... | .02 | .. | .. | .08 | |
| | 30 000 | .... | .03 | .. | .. | .08 | |
| | 31 000 | .... | .03 | .. | .. | .09. | |
| | 31 800 | 31 020 | .. | .. | .. | .. | |
| | 9 400 | 9 170 | .. | .. | .. | .19 | |
| | 6 500 | .... | .90 | .. | .. | .. | |
| | 200 | .... | .57 | .. | .. | .15 | Max. deviation at 18 in. |
| | 1 000 | .... | .57 | .. | .. | .. | do. do. |
| | 2 000 | .... | .60 | .. | .. | .. | do. do. |
| | 3 000 | .... | .64 | .. | .. | .. | do. do. |
| | 4 000 | .... | .67 | .. | .. | .. | do. do. |
| | 5 000 | .... | .72 | .. | .. | .. | do. do. |
| | 6 000 | .... | .78 | .. | .. | .. | do. do. |
| | 7 000 | .... | .87 | .. | .. | .. | do. do. |
| | | | | | | | Extent of rocking motion. Specimen removed. |

Ordinate south, maximum at 18 inches = .60 inch.
" west, maximum " = .12 "
Chord shortening..... = .04 "

Shortening Per Inch.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| North Side | 1 to 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 to 38 |
| | .0 | .01 | .01 | .01 | .01 | .01 | .01 | .005 | .005 | 0 |

South Side.—Lengthening not perceptible.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 14· | 00 | 00 | 00 | .. | .. | .03 | |
| | 200 | .... | 00 | .. | .. | .03 | |
| Length, 40.00 in. | 5 000 | .... | 00 | .. | .. | .02 | |
| | 10 000 | .... | .01 | .. | .. | .02 | |
| Size, | 15 000 | .... | .03 | .. | .. | .03 | |
| 1.006 x 1.000 in. | 16 000 | .... | .03 | .. | .. | .03 | |
| | 17 000 | .... | .04 | .. | .. | .03 | |
| | 18 000 | .... | .05 | .. | .. | .03 | |
| | 19 000 | 18 890 | .07 | .. | .. | .. | Went rather suddenly. |
| | 11 500 | 11 430 | .49 | .. | .. | .01 | |
| | 200 | .... | .11 | .. | .. | .. | |
| | 1 000 | .... | .13 | .. | .. | .. | |
| | 2 000 | .... | .13 | .. | .. | .. | |
| | 3 000 | .... | .14 | .. | .. | .. | |
| | 4 000 | .... | .14 | .. | .. | .. | |
| | 5 000 | .... | .16 | .. | .. | .. | |
| | 6 000 | .... | .17 | .. | .. | .. | |
| | 7 000 | .... | .19 | .. | .. | .. | |
| | 8 000 | .... | .21 | .. | .. | .. | |
| | 9 000 | .... | .25 | .. | .. | .. | |
| | 10 000 | .... | .30 | .. | .. | .. | |
| | 11 000 | .... | .37 | .. | .. | .. | |
| | 11 700 | 11 630 | .56 | .. | .. | .. | |
| | 10 600 | .... | .59 | .. | .. | .. | |
| | 200 | .... | .17 | .. | .. | 00 | Removed specimen. |

Ordinate south, at middle = .22 inch.
" east " = .03 "
Chord shortening..... = .01 "

Shortening Per Inch.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| South Side | 1 to 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 to 40 |
| | 0 | .0017 | .0017 | .0017 | .0017 | .0017 | .0017 | 0 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | | |
| Mark, K 5· | 00 | 00 | .. | 00 | 00 | .. | |
| | 200 | .... | .. | 00 | 00 | .. | |
| Length, 40.01 in. | 5 000 | .... | .. | 00 | 00 | .. | |
| | 10 000 | .... | .. | 00 | 00 | .. | |
| Size. | 15 000 | .... | .. | .01 | .01 | .. | |
| 1.005 x 1.000 in. | 20 000 | .... | .. | .01 | .01 | .. | |
| | 21 000 | .... | .. | .01 | .01 | .. | |
| Right hand twist = | 22 000 | .... | .. | .01 | .01 | .. | |
| .10 in. | 23 000 | .... | .. | .01 | .01 | .. | |
| | 24 000 | .... | .. | 00 | .01 | .. | |
| | 25 000 | .... | .. | 00 | .01 | .. | |
| | 26 000 | .... | .. | 00 | .01 | .. | |
| | 27 000 | .... | .. | 00 | .01 | .. | |
| | 28 000 | .... | .. | 00 | .01 | .. | |
| | 29 000 | .... | .. | 00 | .01. | .. | |
| | 30 000 | .... | .02 | .. | .01 | .. | |
| | 31 000 | 30 600 | .04 | .. | .. | .. | Went suddenly. |
| | 9 000 | 8 950 | .88 | .. | .. | .. | |
| | 200 | .... | .43 | .. | .02 | .. | |
| | 1 000 | .... | .43 | .. | .. | .. | |
| | 2 000 | .... | .46 | .. | .. | .. | |
| | 3 000 | .... | .50 | .. | .. | .. | |
| | 4 000 | ... | .54 | .. | .. | .. | |
| | 5 000 | .... | .59 | .. | .. | .. | |
| | 6 000 | .... | .65 | .. | .. | .. | |
| | 7 000 | .... | .74 | .. | .. | .. | |
| | 8 000 | .... | .83 | .. | .. | .. | |
| | 9 000 | .... | .89 | .. | .. | .. | Extent of rocking motion. |
| | 200 | .... | .44 | .. | .. | .. | Removed specimen. |

Ordinate south at middle = .46 inch.
" west " = .05 "
Chord shortening..... = .01 "

Shortening Per Inch.

| | | | | | | |
|---|---|---|---|---|---|---|
| South Side | 1 to 18 | 19 | 20 | 21 | .22 | 23 to 40 |
| | 0 | .005 | .005 | .005 | .005 | 0 |

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | | In. | |
| Mark, K 6· | 00 | 00 | 00 | .. | .. | .04 | |
| | 500 | .... | 00 | .. | .. | .04 | |
| Length, 42 01 in. | 5 000 | .... | 00 | .. | .. | .04 | |
| | 10 000 | .... | .01 | .. | .. | .04 | |
| Size, | 15 000 | .... | .02 | .. | .. | .05 | |
| .997 x 1.005 in. | 16 000 | .... | .02 | .. | .. | .05 | |
| | 17 000 | .... | .03 | .. | .. | .05 | |
| | 18 000 | 18 000 | .04 | .. | .. | .. | Gone. |
| | 12 700 | 12 700 | .43 | .. | .. | .. | Rockers free. |
| | 500 | .... | .02 | .. | .. | .04 | |
| | 5 000 | .... | .03 | .. | .. | .04 | |
| | 10 000 | .... | .06 | .. | .. | .04 | |
| | 11 000 | .... | .07 | .. | .. | .04 | |
| | 12 000 | .... | .10 | .. | .. | .05 | |
| | 13 000 | 13 000 | .. | .. | .. | .. | Going. |
| | 12 700 | .... | .33 | .. | .. | .. | |
| | 500 | .... | .03 | .. | .. | .04 | Removed specimen. |

Ordinate south at middle = .03 inch.
" east " = .03 "
Chord shortening..... = .01 "
Shortening per inch not perceptible.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | | In. | | |
| Mark, K 6: | 00 | 00 | .04 | .. | .04 | .. | |
| | 500 | .... | .04 | .. | .04 | .. | |
| Length, 42.01 in. | 5 000 | .... | .05 | .. | .05 | .. | |
| | 10 000 | .... | .06 | .. | .05 | .. | |
| Size, | 11 000 | .... | .07 | .. | .05 | .. | |
| 1.007 x 1.000 in. | 12 000 | .... | .07 | .. | .05 | .. | |
| | 13 000 | .... | .08 | .. | .05 | .. | |
| | 14 000 | .... | .09 | .. | .05 | .. | |
| | 14 700 | 14 600 | .. | .. | .. | .. | Went very slowly. |
| | 13 000 | 12 910 | .39 | .. | .. | .. | |
| | 500 | .... | .05 | .. | .04 | .. | |
| | 500 | .... | .05 | .. | .04 | .. | Placed .03 in. south of axis. |
| | 5 000 | .... | .05 | .. | .05 | .. | |
| | 10 000 | .... | .06 | .. | .05 | .. | |
| | 11 000 | .... | .06 | .. | .05 | .. | |
| | 12 000 | .... | .06 | .. | .05 | .. | |
| | 13 000 | .... | .07 | .. | .05 | .. | |
| | 14 000 | .... | .07 | .. | .05 | .. | |
| | 15 000 | .... | .07 | .. | .05 | .. | |
| | 16 000 | .... | .07 | .. | .05 | .. | |
| | 17 000 | .... | .08 | .. | .05 | .. | |
| | 18 000 | .... | .08 | .. | .05 | .. | |
| | 19 000 | .... | .08 | .. | .04 | .. | |
| | 20 000 | .... | .09 | .. | .04 | .. | |
| | 21 000 | .... | .10 | .. | .04 | .. | |
| | 22 000 | 21 850 | .11 | .. | .. | .. | Gone suddenly. Rocker free. |
| | 10 800 | 10 730 | .74 | .. | .. | .. | |
| | 500 | .... | .22 | .. | .05 | .. | |
| | 5 000 | .... | .27 | .. | .05 | .. | |
| | 6 000 | .... | .29 | .. | .. | .. | |
| | 7 000 | .... | .32 | .. | .. | .. | |
| | 8 000 | .... | .37 | .. | .. | .. | |
| | 9 000 | .... | .43 | .. | .01 | .. | |
| | 10 000 | .... | .56 | .. | .. | .. | |
| | 10 700 | 10 630 | .. | .. | .. | .. | Fails. |
| | 10 000 | .... | .82 | .. | .. | .. | Scale drops on south side. |
| | 500 | .... | .30 | .. | .03 | .. | Removed specimen. |

Ordinate south at middle = .29 inch.
" west " = .03 "
Chord shortening..... = .01 "
Scale disturbed over middle 10 inches on south side.
Middle 8 inches on south side shorten .015 inch.
Middle 8 inches on north side apparently lengthened .01 inch.

| Description of Specimen. | Load. Lbs. | Load. Lbs per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | | |
| Mark, K 7· | 00 | 00 | .. | .07 | .. | 00 | |
| | 500 | .... | .. | .07 | .. | 00 | |
| Length, 44 04 in. | 5 000 | .... | .. | .10 | .. | 00 | |
| | 10 000 | .... | .. | .23 | .. | 00 | |
| Size, | 11 000 | .... | .. | .37 | .. | .. | Going slowly. |
| .995 x 1.002 in. | 11 000 | .... | .. | .43 | .. | .. | |
| | 11 000 | 11 000 | .. | .47 | .. | .. | Fails. |
| | 9 400 | .... | .. | .66 | .. | .. | |
| | 500 | .... | .. | .18 | .. | 00 | |
| | 500 | .... | .. | .17 | .. | .. | By applying pressure in direction to diminish deviation and releasing pressure. |
| | 5 000 | .... | .. | .25 | .. | .. | |
| | 6 000 | .... | .. | .29 | .. | .. | |
| | 7 000 | .... | .. | .44 | .. | .. | |
| | 8 000 | .... | .. | .51 | .. | .. | |
| | 9 000 | .... | .. | .54 | .. | .. | |
| | 9 600 | 9 600 | .. | .75 | .. | .. | Fails slowly. |
| | 9 000 | .... | .. | .77 | .. | .. | |
| | 500 | .... | .. | .24 | .. | 00 | Specimen removed. |

Ordinate north, at middle..... = .24 inch.
" west, " ..... = .01 "
Chord shortening..... = .01 "
Shortening in middle four inches north side. = .01 "

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | In. | | |
| Mark, K 7: | 00 | 00 | .. | .03 | .02 | .. | |
| | 500 | .... | .. | .03 | .02 | .. | |
| Length, 44.04 in. | 5 000 | .... | .. | .03 | .03 | .. | |
| | 10 000 | .... | .. | .03 | .03 | .. | |
| Size, | 11 000 | .... | .. | .03 | .03 | .. | |
| 1.002 x 995 in. | 12 000 | .... | .. | .03 | .03 | .. | |
| | 13 000 | .... | .. | .03 | .03 | .. | |
| Specimen curved | 14 000 | .... | .. | .03 | .03 | .. | |
| east and west like | 15 000 | .... | .. | .03 | .03 | .. | |
| letter S. | 16 000 | .... | .. | .03 | .03 | .. | |
| | 17 000 | .... | .. | .03 | .03 | .. | |
| Ordinates | 18 000 | .... | .. | .03 | .03 | .. | |
| west at | 19 000 | .... | .. | .04 | .03 | .. | |
| 0....=.06 in. | 20 000 | .... | .. | .04 | .03 | .. | |
| 11....=.00 " | 21 000 | .... | .. | .04 | .03 | .. | |
| 22....=.02 " | 22 000 | .... | .. | .04 | .03 | .. | |
| 33....=.03 " | 23 000 | .... | .. | .04 | .04 | .. | |
| 44....=.0 " | 24 000 | .... | .. | .04 | .04 | .. | |
| | 25 000 | .... | .. | .04 | .04 | .. | |
| | 26 000 | .... | .. | .04 | .04 | .. | |
| | 27 000 | .... | .. | .05 | .05 | .. | |
| | 28 000 | 28 000 | .. | .08 | .. | .. | Failed. |
| | 8 000 | .... | .. | .97 | .. | .. | Extent of rocking motion. |
| | 500 | .... | .. | .46 | .. | .. | |
| | 1 000 | .... | .. | .48 | .. | .. | |
| | 2 000 | .... | .. | .49 | .. | .. | |
| | 3 000 | .... | .. | .55 | .. | .. | |
| | 4 000 | .... | .. | .61 | .. | .. | |
| | 5 000 | .... | .. | .67 | .. | .. | |
| | 6 000 | .... | .. | .77 | .. | .. | |
| | 7 000 | .... | .. | .90 | .. | .. | Extent of motion. |
| | 500 | .... | .. | .47 | .. | .. | Removed specimen. |

After Specimen Removed:

Ordinate north at middle..... = .45 inch.
" west at 0..... = .07 "
" " 11..... = 0 "
" " 22..... = .03 "
" " 33..... = .04 "
" " 44..... = 0 "
Chord shortening..... = .02 "
Shortening in middle six inches, north side = .015 "

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In | | In. | | |
| Mark, K 8· | 00 | 00 | 00 | .. | .01 | .. | |
| | 500 | .... | 00 | .. | .01 | .. | |
| Length, 48.00 in. | 5 000 | .... | .01 | .. | .01 | .. | |
| | 10 000 | .... | .02 | .. | .01 | .. | |
| Size, | 14 200 | 14 120 | .. | .. | .. | .. | Failed. |
| 1 000 x 1.006 in. | 10 700 | 10 640 | .44 | .. | .. | .. | |
| | 500 | .... | .01 | .. | .01 | . | |
| | 500 | .... | 00 | .. | .01 | .. | Placed .01 in. south of axis. |
| | 5 000 | .... | .01 | .. | .01 | .. | |
| | 10 000 | .... | .02 | .. | .02 | .. | |
| | 11 000 | .... | .03 | .. | 00 | .. | |
| | 12 000 | .... | .03 | .. | 00 | .. | |
| | 13 000 | .... | .03 | .. | 00 | .. | |
| | 14 000 | .... | .03 | .. | 00 | .. | |
| | 15 000 | .... | .04 | .. | 00 | .. | |
| | 16 000 | .... | .04 | .. | 00 | .. | |
| | 17 000 | .... | .04 | .. | 00 | .. | |
| | 18 000 | .... | .05 | .. | 00 | .. | |
| | 18 600 | 18 490 | .. | .. | .. | .. | Failed. |
| | 9 600 | 9 550 | .69 | .. | .. | .. | Rocker free. |
| | 500 | .... | .06 | .. | .02 | .. | |
| | 5 000 | .... | .08 | .. | .. | .. | |
| | 6 000 | .... | .09 | .. | .. | .. | |
| | 7 000 | .... | .10 | .. | .. | .. | |
| | 8 000 | .... | .11 | .. | .. | .. | |
| | 9 000 | .... | .12 | .. | .. | .. | |
| | 10 000 | .... | .16 | .. | .. | .. | |
| | 10 700 | 10 640 | .. | .. | .. | .. | Failed. |
| | 10 200 | .... | .43 | .. | .. | .. | |
| | 500 | .... | .06 | .. | .01 | .. | Specimen removed. |

Ordinate north at middle..... = .06 inch.
" east " ..... = .01 "
Chord shortening..... = .00 "
Shortening per inch not perceptible.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In. | In. | |
| Mark, K 9· | 00 | 00 | .. | .05 | .04 | .. | |
| | 500 | .... | .. | .05 | .04 | .. | |
| Length, 52 00 in. | 1 000 | .... | .. | .05 | .04 | .. | |
| | 2 000 | .... | .. | .05 | .04 | .. | |
| Size, | 3 000 | .... | .. | .05 | .04 | .. | |
| .995 x 1.005 in. | 4 000 | .... | .. | .05 | .04 | .. | |
| | 5 000 | .... | .. | .06 | .04 | .. | |
| | 6 000 | .... | .. | .06 | .01 | .. | |
| | 7 000 | .... | .. | .06 | .04 | .. | |
| | 8 000 | .... | .. | .07 | .04 | .. | |
| | 9 000 | .... | .. | .07 | .04 | .. | |
| | 10 000 | .... | .. | .08 | .04 | .. | |
| | 11 000 | .... | .. | .09 | .04 | .. | |
| | 12 000 | .... | .. | .09 | .04 | .. | |
| | 13 000 | .... | .. | .09 | .04 | .. | |
| | 14 000 | .... | .. | .11 | .04 | .. | |
| | 15 000 | 15 000 | .. | .. | .. | .. | Failed. |
| | 8 300 | 8 300 | .. | .68 | .. | .. | |
| | 500 | .... | .. | .08 | .04 | .. | |
| | 5 000 | .... | 00 | 00 | .04 | .. | While load is rising to 5 000, pressure is put on rocker in direction to correct deviation. |

(*Mark K 9· continued.*)

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | | |
| Mark, K 9· | 10 000 | .... | 00 | .. | .04 | .. | |
| | 11 000 | .... | 00 | .. | .. | .. | |
| Length, 52.00 in. | 12 000 | .... | .01 | .. | .. | .. | |
| | 13 000 | .... | .01 | .. | .. | .. | |
| Size. | 14 000 | .... | .01 | .. | .. | .. | |
| .995 x 1.005 in. | 15 000 | .... | .02 | .. | .. | .. | |
| | 16 000 | .... | .02 | .. | .. | .. | |
| | 17 000 | .... | .02 | .. | .. | .. | |
| | 18 000 | .... | .03 | .. | .. | .. | |
| | 19 000 | .... | .04 | .. | .. | .. | Going. |
| | 19 000 | .... | .06 | .. | .. | .. | |
| | 19 700 | 19 700 | .. | .. | .. | .. | Failed. |
| | 8 500 | 8 500 | .82 | .. | .. | .. | Rockers free. |
| | 500 | .... | .04 | .. | .04 | .. | |
| | 5 000 | .... | .06 | .. | .03 | .. | While load is rising to 5 000, pressure is put on rockers in direction to correct deviation. |
| | 6 000 | .... | .07 | .. | .. | .. | |
| | 7 000 | .... | .08 | .. | .. | .. | |
| | 8 000 | .... | .09 | .. | .. | .. | |
| | 9 000 | .... | .15 | .. | .. | .. | |
| | 9 600 | 9 600 | .. | .. | .. | .. | Failed. |
| | 8 700 | 8 700 | .37 | .. | .. | .. | |
| | 500 | .... | .04 | .. | .. | .. | |
| | 5 000 | .... | .. | .01 | .. | .. | By pressure on rockers while load rises to 4 000. |
| | 6 000 | .... | .. | .01 | .. | .. | |
| | 7 000 | .... | .. | .01 | .. | .. | |
| | 8 000 | .... | .. | .01 | .. | .. | |
| | 9 000 | .... | .. | .. | .. | .. | |
| | 10 000 | .... | .. | .. | .. | .. | |
| | 11 000 | .... | .01 | .. | .. | .. | |
| | 12 000 | .... | .01 | .. | .. | .. | |
| | 13 000 | .... | .02 | .. | .. | .. | |
| | 14 000 | .... | .03 | .. | .. | .. | |
| | 14 900 | .... | .. | .. | .. | .. | Gone. Same direction as above. |
| | 10 700 | .... | .. | .. | .. | .. | |
| | 500 | .... | .04 | .. | .. | .. | |
| | 10 000 | .... | .. | .33 | .. | .. | By pressure on rockers while load rises to 10 000. |
| | 6 500 | .... | .. | .09 | .. | .. | |
| | 10 000 | .... | .. | .10 | .. | .. | Releasing pressure while load drops to 6 500. |
| | 11 000 | .... | .. | .11 | .. | .. | |
| | 12 000 | .... | .. | .11 | .. | .. | |
| | 13 000 | .... | .. | .12 | .. | .. | |
| | 14 000 | .... | .. | .13 | .. | .. | |
| | 15 000 | .... | .. | .14 | .. | .. | |
| | 16 000 | .... | .. | .15 | .. | .. | |
| | 16 900 | 16 900 | .. | .. | .. | .. | Failed. |
| | 8 200 | .... | .. | .78 | .. | .. | |
| | 500 | .... | .. | ·09 | .04 | .. | Specimen removed. |

After Removing Specimen.

Ordinate north at middle = .06 inches.
" west " = .03 inch.
Chord shortening ..... = 00 "
Scale undisturbed.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | In. | |
| Mark, K 9: | 00 | 00 | .. | .06 | .. | .05 | |
| | 500 | .... | .. | .06 | .. | .05 | |
| Length, 52.00 in. | 5 000 | .... | .. | .07 | .. | .05 | |
| | 6 000 | .... | .. | .08 | .. | .. | |
| Size, | 7 000 | .... | .. | .08 | .. | .05 | |
| .995 x 1.005 in. | 8 000 | .... | .. | .09 | .. | .. | |
| | 9 000 | .... | .. | .10 | .. | .. | |
| | 10 000 | .... | .. | .11 | .. | .. | |
| | 11 000 | .... | .. | .12 | .. | .. | |
| | 12 000 | 12 000 | .. | .. | .. | .. | Failed. |
| | 9 100 | 9 100 | .. | .46 | .. | .. | |
| | 500 | .... | .. | .07 | .. | .05 | |
| | 500 | .... | .. | .06 | .. | .05 | Specimen placed .06 in north of axis. |
| | 5 000 | .... | .. | .04 | .. | .05 | |
| | 6 000 | .... | .. | .03 | .. | .. | |
| | 7 000 | .... | .. | .02 | .. | .. | |
| | 8 000 | .... | .02 | .. | .. | .. | |
| | 8 700 | 8 700 | .. | .. | .. | .. | Went very slowly. |
| | 8 500 | 8 500 | .30 | .. | .. | .. | |
| | 500 | .... | .. | .05 | .. | .. | |
| | 500 | .... | .. | .06 | .. | .05 | Specimen placed .04 in. north of axis. |
| | 5 000 | .... | .. | .05 | .. | .05 | |
| | 6 000 | .... | .. | .05 | .. | .05 | |
| | 7 000 | .... | .. | .04 | .. | .05 | |
| | 8 000 | .... | .. | .04 | .. | .. | |
| | 9 000 | .... | .. | .04 | .. | .. | |
| | 10 000 | .... | .. | .04 | .. | .. | |
| | 11 000 | .... | .. | .03 | .. | .06 | |
| | 12 000 | 12 000 | .. | .. | .. | .. | Failed. |
| | 8 800 | 8 800 | .35 | .. | .. | .. | |
| | 500 | .... | .. | .05 | .. | .05 | |
| | 500 | .... | .. | .06 | .. | .05 | Specimen placed .02 in. north of axis. |
| | 5 000 | .... | .. | .06 | .. | .05 | |
| | 7 000 | .... | .. | .06 | .. | .. | |
| | 8 000 | .... | .. | .07 | .. | .. | |
| | 9 000 | .... | .. | .07 | .. | .. | |
| | 10 000 | .... | .. | .08 | .. | .. | |
| | 11 000 | .... | .. | .08 | .. | .. | |
| | 12 000 | .... | .. | .08 | .. | .. | |
| | 13 000 | .... | .. | .09 | .. | .. | |
| | 14 000 | .... | .. | .09 | .. | .. | |
| | 15 000 | .... | .. | .10 | .. | .. | |
| | 16 000 | .... | .. | .11 | .. | .. | |
| | 17 000 | .... | .. | .12 | .. | .. | |
| | 18 000 | .... | .. | .14 | .. | .. | |
| | 18 200 | 18 200 | .. | .. | .. | .. | Failed. |
| | 8 400 | 8 400 | .. | .89 | .. | .. | Rockers free. |
| | 500 | .... | .. | .13 | .. | .05 | Specimen removed. |

Ordinate north at middle = .13 inch.
" east " = .05 "
Chord Shortening..... = 00 "

Scale very slightly disturbed middle of north side.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | In | | |
| Mark, K 10: | 00 | 00 | .. | .01 | .07 | .. | |
| | 500 | .... | .. | .01 | .07 | .. | |
| Length, 56.01 in. | 5 000 | .... | .. | .02 | .05 | .. | |
| | 6 000 | .... | .. | .02 | .05 | .. | |
| Size, | 7 000 | .... | .. | .02 | .05 | .. | |
| .998 x 1.002 in. | 8 000 | .... | .. | .02 | .05 | .. | |
| | 9 000 | .... | .. | .02 | .05 | .. | |
| | 10 000 | .... | .. | .02 | .05 | .. | |
| | 11 000 | .... | .. | .03 | .05 | .. | |
| | 12 000 | .... | .. | .03 | .05 | .. | |
| | 13 000 | .... | .. | .04 | .06 | .. | |
| | 14 000 | .... | .. | .04 | .06 | .. | |
| | 15 000 | .... | .. | .05 | .06 | .. | |
| | 16 000 | .... | .. | .06 | .07 | .. | |
| | 17 000 | .... | .. | .07 | .07 | .. | |
| | 17 500 | 17 500 | .. | .. | .. | .. | Failed. |
| | 7 600 | 7 600 | .. | .86 | .. | .. | Went very suddenly. |
| | 500 | .... | .. | .06 | .07 | .. | |
| | 5 000 | .... | .04 | .. | .06 | .. | By pressure on rockers in direction to correct deviation and while load rises to 5 000. |
| | 6 000 | .... | .05 | .. | .05 | .. | |
| | 7 000 | .... | .05 | .. | .05 | .. | |
| | 8 000 | .... | .05 | .. | .06 | .. | |
| | 9 000 | .... | .06 | .. | .06 | .. | |
| | 10 000 | .... | .06 | .. | .06 | .. | |
| | 11 000 | .... | .07 | .. | .06 | .. | |
| | 12 000 | .... | .08 | .. | .06 | .. | |
| | 13 000 | .... | .09 | .. | .07 | .. | |
| | 13 900 | 13 900 | .. | .. | .. | .. | Failed. |
| | 7 500 | 7 500 | .69 | .. | .. | .. | Rockers free. |
| | 500 | .... | .01 | .. | .07 | .. | |
| | 500 | .... | .. | .01 | .07 | .. | |
| | 5 000 | .... | .. | .01 | .05 | .. | |
| | 6 000 | .... | .. | .02 | .05 | .. | |
| | 7 000 | .... | .. | .02 | .05 | .. | |
| | 8 000 | .... | .. | .02 | .05 | .. | |
| | 9 000 | .... | .. | .02 | .05 | .. | |
| | 10 000 | .... | .. | .03 | .05 | .. | |
| | 11 000 | .... | .. | .03 | .05 | .. | |
| | 12 000 | .... | .. | .04 | .05 | .. | |
| | 13 000 | .... | .. | .04 | .06 | .. | |
| | 14 000 | .... | .. | .04 | .06 | .. | |
| | 15 000 | .... | .. | .05 | .06 | .. | |
| | 16 000 | .... | .. | .06 | .07 | .. | |
| | 17 000 | 17 000 | .. | .. | .. | .. | Failed. |
| | 7 400 | 7 400 | .. | .83 | .. | .. | |
| | 500 | .... | .. | .06 | .07 | .. | |
| | 5 000 | .... | .. | .08 | .06 | .. | |
| | 6 000 | .... | .. | .10 | .06 | .. | |
| | 7 000 | .... | .. | .11 | .06 | .. | |
| | 8 000 | .... | .. | .12 | .06 | .. | |
| | 9 000 | .... | .. | .13 | .05 | .. | |
| | 9 900 | .... | .. | .. | .. | .. | |
| | 7 900 | .... | .. | .42 | .. | .. | |
| | 500 | .... | .. | .06 | .08 | .. | |
| | 5 000 | .... | .. | .04 | .. | .. | By pressure on rockers while load rises to 5 000. |
| | 6 000 | .... | .. | .04 | .06 | .. | |
| | 7 000 | .... | .. | .05 | .06 | .. | |
| | 8 000 | .... | .. | .05 | .06 | .. | |
| | 9 000 | .... | .. | .06 | .06 | .. | |
| | 10 000 | .... | .. | .06 | .06 | .. | |
| | 11 000 | .... | .. | .07 | .06 | .. | |

(*Mark K* 10: *continued.*)

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. | | | | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | N. | S. | E. | W. | |
| Mark, K 10: | | | | In. | In. | | |
| Length, 56.01 in. | 12 000 | .... | .. | .08 | .06 | .. | |
| | 13 000 | 13 000 | .. | .11 | .. | .. | Failed. |
| Size, | 6 900 | 6 900 | .. | .68 | .. | .. | |
| .998 x 1.002 in. | 500 | .... | .. | .06 | .08 | .. | Specimen removed. |

Before Testing.

Ordinates north { 14 28 42 56 inches.
{ .02 .01 0 0 inch.

" west..... { 28 42 14 inches.
{ .07 .11 .02 inch.

After Specimen Removed.

Ordinates north { 14 28 42 inches.
{ .04 .05 .02 inch.

" west. { 14 28 42 inches.
{ .02 .07 .10 inch.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. | | | | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | N. | S. | E. | W. | |
| | | | In. | | | In. | |
| Mark, K 10· | 00 | 00 | .04 | .. | .. | .08 | |
| | 500 | .... | .04 | .. | .. | .08 | |
| Length, 56.01 in. | 5 000 | .... | .08 | .. | .. | .08 | |
| | 6 000 | .... | .09 | .. | .. | .08 | |
| Size, | 7 000 | .... | .11 | .. | .. | .09 | |
| .998 x 1.002 in. | 8 000 | 8 000 | .. | .. | .. | .. | Went very slowly. |
| | 7 500 | 7 500 | .47 | .. | .. | .11 | |
| | 500 | .... | .06 | .. | .. | .09 | |
| | 5 000 | .... | .09 | .. | .. | .09 | |
| | 6 000 | .... | .10 | .. | .. | .09 | |
| | 7 000 | .... | .13 | .. | .. | .09 | |
| | 8 000 | 8 000 | .23 | .. | .. | .. | Went very slowly. |
| | 7 500 | 7 500 | .44 | .. | .. | .11 | |
| | 500 | .... | .06 | .. | .. | .08 | Specimen removed. |

Ordinate south at middle = .04 inch.
" east " = .08 "

No perceptible scaling.

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | In. | In. | | In. | |
| Mark, K 20· | 00 | 00 | .. | .02 | .. | .04 | |
| | 500 | .... | .. | .02 | .. | .04 | |
| Length, 60.00 in. | 5 000 | .... | .. | .02 | .. | .06 | |
| | 6 000 | .... | .. | .02 | .. | .07 | |
| Size, | 7 000 | .... | .. | .01 | .. | .07 | |
| 1.005 x .995 in. | 8 000 | .... | .. | 00 | .. | .07 | |
| | 9 000 | .... | .. | 00 | .. | .07 | |
| | 10 000 | .... | .. | 00 | .. | .08 | |
| | 11 000 | .... | .01 | .. | .. | .09 | |
| | 12 000 | .... | .02 | .. | .. | .10 | |
| | 12 800 | 12 800 | .. | .. | .. | .. | Went rather suddenly. |
| | 6 400 | 6 400 | .68 | .. | .. | .11 | |
| | 500 | .... | 00 | .. | .. | .05 | |
| | 8 000 | .... | .06 | .. | .. | .08 | |
| | 9 000 | .... | .08 | .. | .. | .10 | |
| | 9 000 | 9 000 | .. | .. | .. | .. | Went rather slowly. |
| | 7 000 | 7 000 | .52 | .. | .. | .11 | |
| | 500 | .... | .01 | .. | .. | .05 | |
| | 5 000 | .... | .03 | .. | .. | .06 | |
| | 6 000 | .... | .03 | .. | .. | .07 | |
| | 7 000 | .... | .04 | .. | .. | .07 | |
| | 8 000 | ... | .06 | .. | .. | .08 | |
| | 9 000 | 9 000 | .. | .. | .. | .. | Failed. |
| | 7 000 | 7 000 | .56 | .. | .. | .. | |
| | 6 800 | .... | .. | .. | .. | .11 | |
| | 500 | .... | .. | .. | .. | .. | |
| | 5 000 | .... | .. | .07 | .. | .06 | By pressure on rockers to cause bar to bend in opposite direction. |
| | 6 000 | .... | . | .08 | .. | .06 | |
| | 7 000 | .... | .. | .07 | .. | .07 | |
| | 8 000 | .... | .. | .08 | .. | .07 | |
| | 9 000 | .... | .. | .08 | .. | .07 | |
| | 10 000 | .... | .. | .08 | .. | .08 | |
| | 11 000 | .... | .. | .09 | .. | .08 | |
| | 12 000 | .... | .. | .10 | .. | .09 | |
| | 13 000 | .... | .. | .11 | .. | .10 | |
| | 14 000 | .... | .. | .13 | .. | .11 | |
| | 14 900 | 14 900 | .. | .. | .. | .. | Failed. |
| | 6 700 | 6 700 | .. | .86 | .. | .. | Rockers free. |
| | 500 | .... | .. | .05 | .. | .04 | |
| | 5 000 | .... | .. | .06 | .. | .06 | |
| | 6 000 | .... | .. | .06 | .. | .06 | |
| | 7 000 | .... | .. | .06 | .. | .07 | |
| | 8 000 | .... | .. | .07 | .. | .07 | |
| | 9 000 | .... | .. | .07 | .. | .07 | |
| | 10 000 | .... | .. | .07 | .. | .08 | |
| | 11 000 | .... | .. | .08 | .. | .09 | |
| | 12 000 | .... | .. | .09 | .. | .09 | |
| | 13 000 | .... | .. | .10 | .. | .09 | |
| | 14 000 | .... | .. | .11 | .. | .11 | |
| | 15 000 | 15 000 | .. | .. | .. | .. | Failed. |
| | 6 400 | 6 400 | .. | .86 | .. | .. | |
| | 500 | ... | .. | .05 | .. | .. | |
| | 500 | .... | .. | .03 | .. | .. | By pressure. |
| | 5 000 | .... | .. | .03 | .. | .06 | |
| | 10 000 | .... | .. | .03 | .. | .08 | |
| | 11 000 | .... | .. | .04 | .. | .09 | |
| | 12 000 | .. . | .. | .04 | .. | .09 | |
| | 13 000 | .... | .. | .04 | .. | .10 | |
| | 14 000 | .... | .. | .04 | .. | .11 | |
| | 15 000 | .... | .. | .04 | .. | .13 | |
| | 16 000 | .... | .. | .04 | .. | .15 | |
| | 17 000 | .... | .. | .04 | .. | .17 | |
| | 18 000 | 18 000 | .. | .. | .. | .. | Started to fail along axis and changed. |
| | 5 400 | 5 400 | .89 | .. | .. | .. | |
| | 500 | .... | .06 | .. | .. | .07 | Removed specimen. |

After Specimen Removed.

Ordinate east at middle, = .06 inch.
" south " = .06 "

| Description of Specimen. | Load. Lbs. | Load. Lbs. per sq. in. | Deviations. N. | S. | E. | W. | Remarks. |
|---|---|---|---|---|---|---|---|
| | | | | In. | | | |
| Mark, K 17· | 00 | 00 | .. | .05 | .. | .06 | |
| | 200 | .... | .. | .05 | .. | .. | |
| Length, 80.00 in. | 500 | .... | .. | .05 | .. | .. | |
| | 1 000 | .... | .. | .05 | .. | .. | |
| Size, | 2 000 | .... | .. | .07 | .. | .. | |
| 1.005 x .995 in. | 3 000 | .... | .. | .07 | .. | .. | |
| | 3 900 | 3 900 | .. | .41 | .. | .. | Failed. |
| | 3 600 | 3 600 | .. | .55 | .. | .. | |
| | 500 | .... | .. | .09 | .. | .. | |
| | 2 000 | .... | .. | .01 | .. | .. | By pressure to bar. |
| | 3 000 | .... | .. | .02 | .. | .. | |
| | 4 000 | .... | .. | .03 | .. | .. | |
| | 5 000 | .... | .. | .04 | .. | .. | |
| | 6 000 | 6 000 | .. | .. | .. | .. | Failed. |
| | 3 100 | 3 100 | .. | .62 | .. | .. | |
| | 500 | .... | .. | .07 | .. | .. | Specimen removed. |

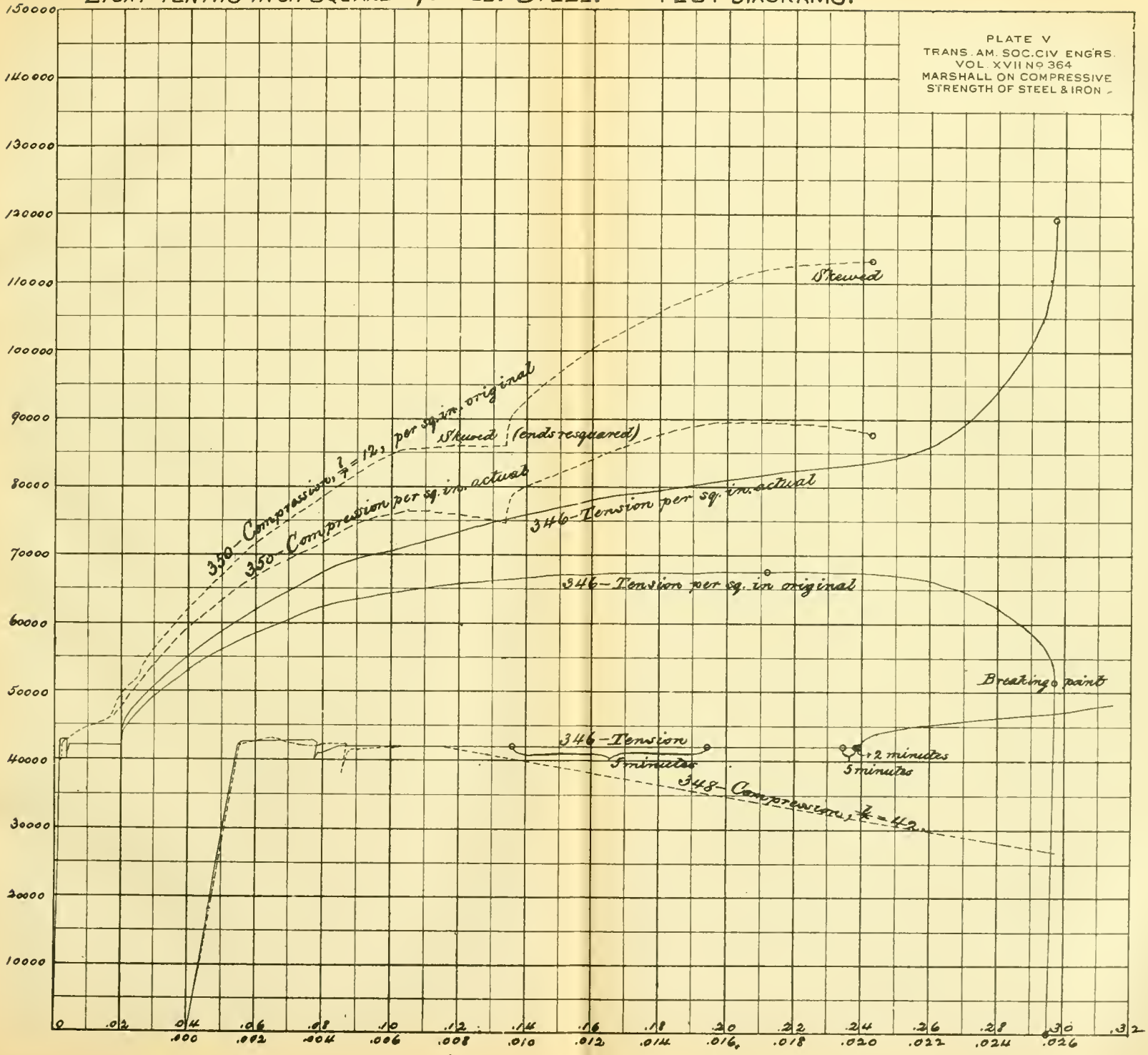
Before Testing,

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Ordinate north | 12 | 24 | 36 | 40 | 44 | 56 | 68 | inches. |
| | .03 | .04 | .05 | .05 | .05 | .03 | .01 | inch. |
| " east.. | 12 | 24 | 36 | 40 | 44 | 56 | 68 | 80 inches. |
| | .02 | .09 | .07 | .06 | .05 | .01 | 0 | —.04 inch. |

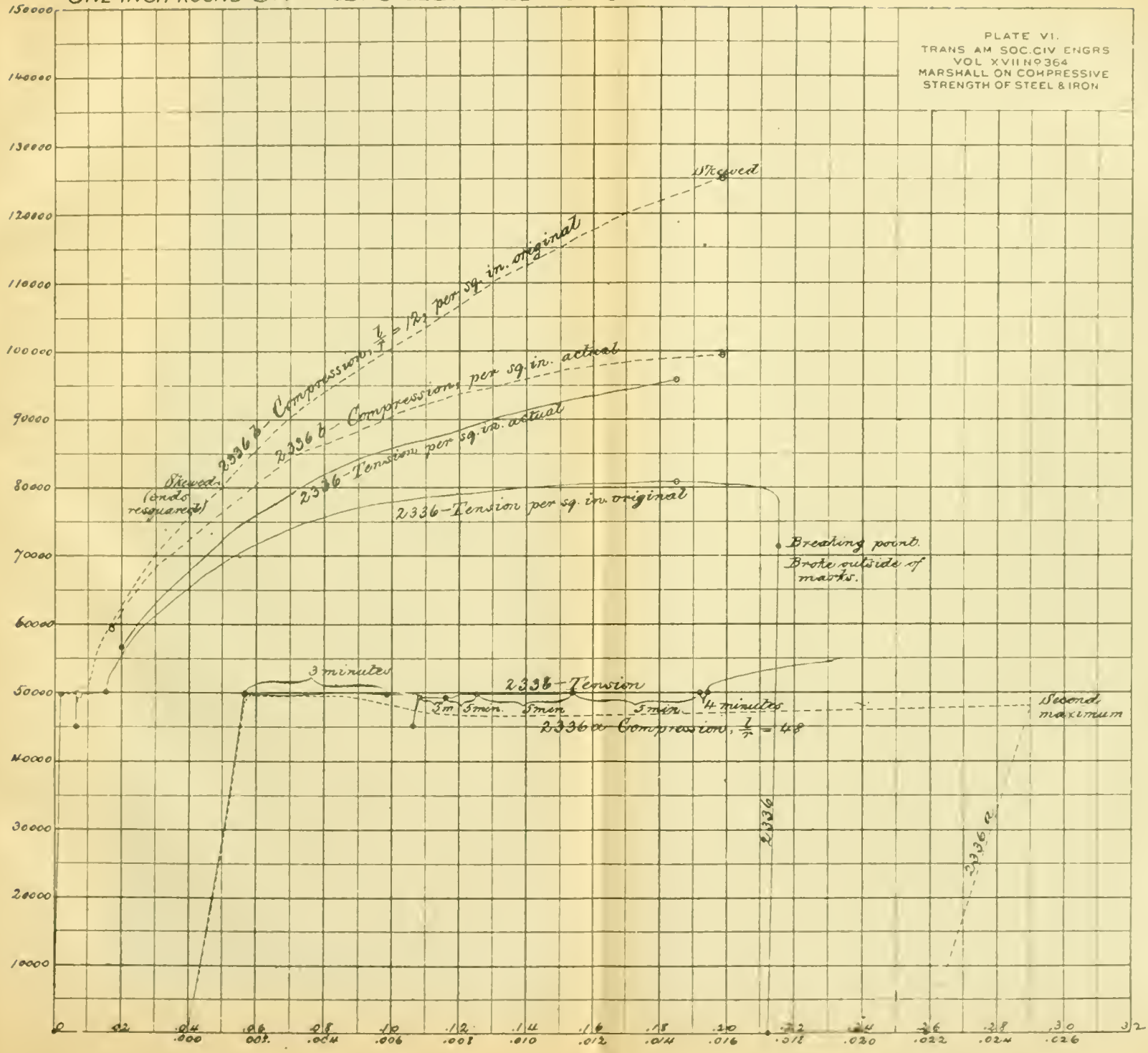
After Removal.

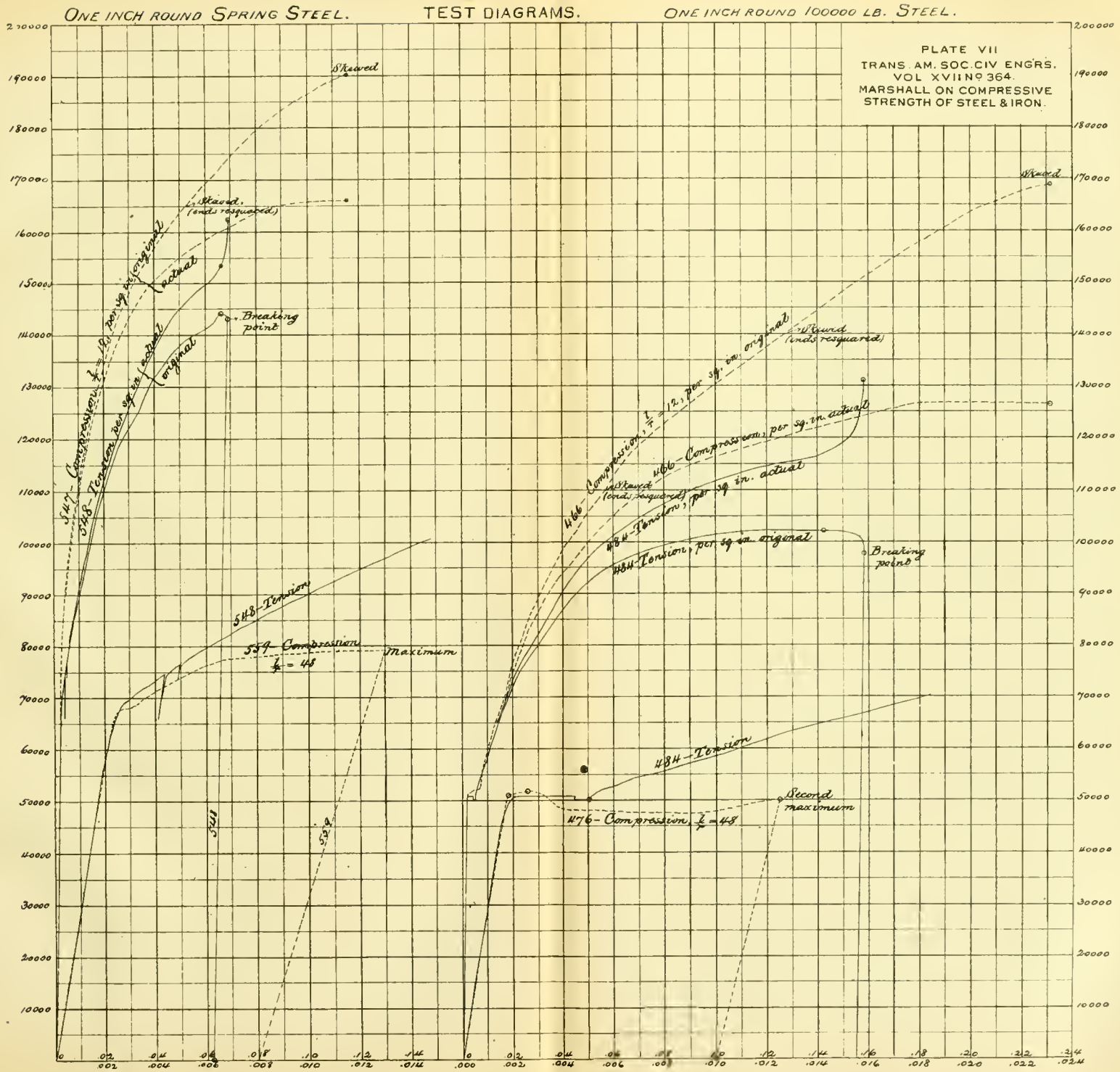
| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Ordinates north | 12 | 24 | 36 | 40 | 44 | 56 | 68 | inches. |
| | .04 | .06 | ·07 | .66 | .05 | .03 | .01 | inch. |
| " east.. | 12 | 24 | 36 | 40 | 44 | 56 | 68 | 80 inches. |
| | .03 | .07 | .07 | .05 | .04 | .01 | 0 | —.04 inch. |

EIGHT TENTHS INCH SQUARE 67000 LB. STEEL. TEST DIAGRAMS.

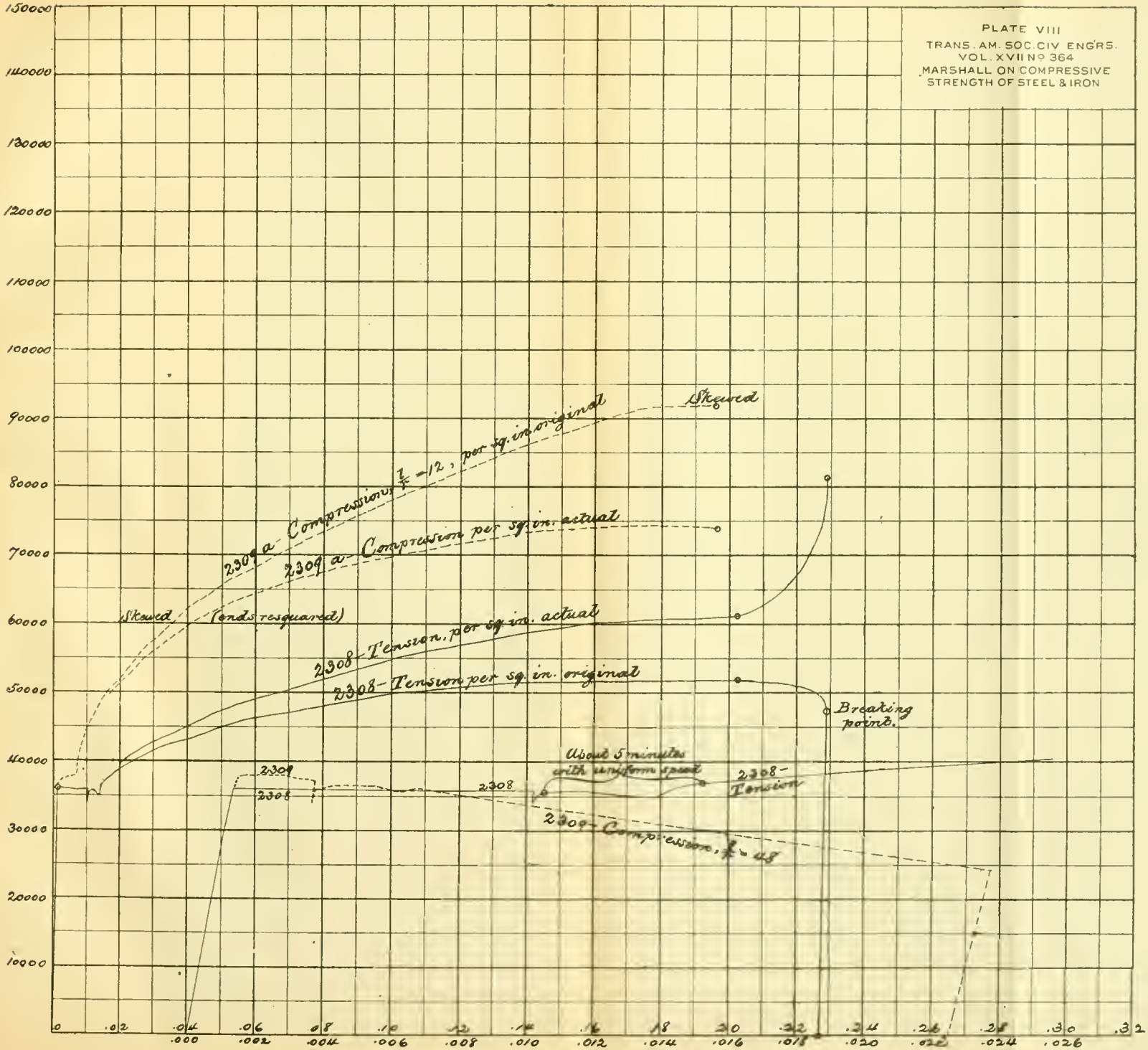


ONE INCH ROUND 80000 LB. STEEL. TEST DIAGRAMS.

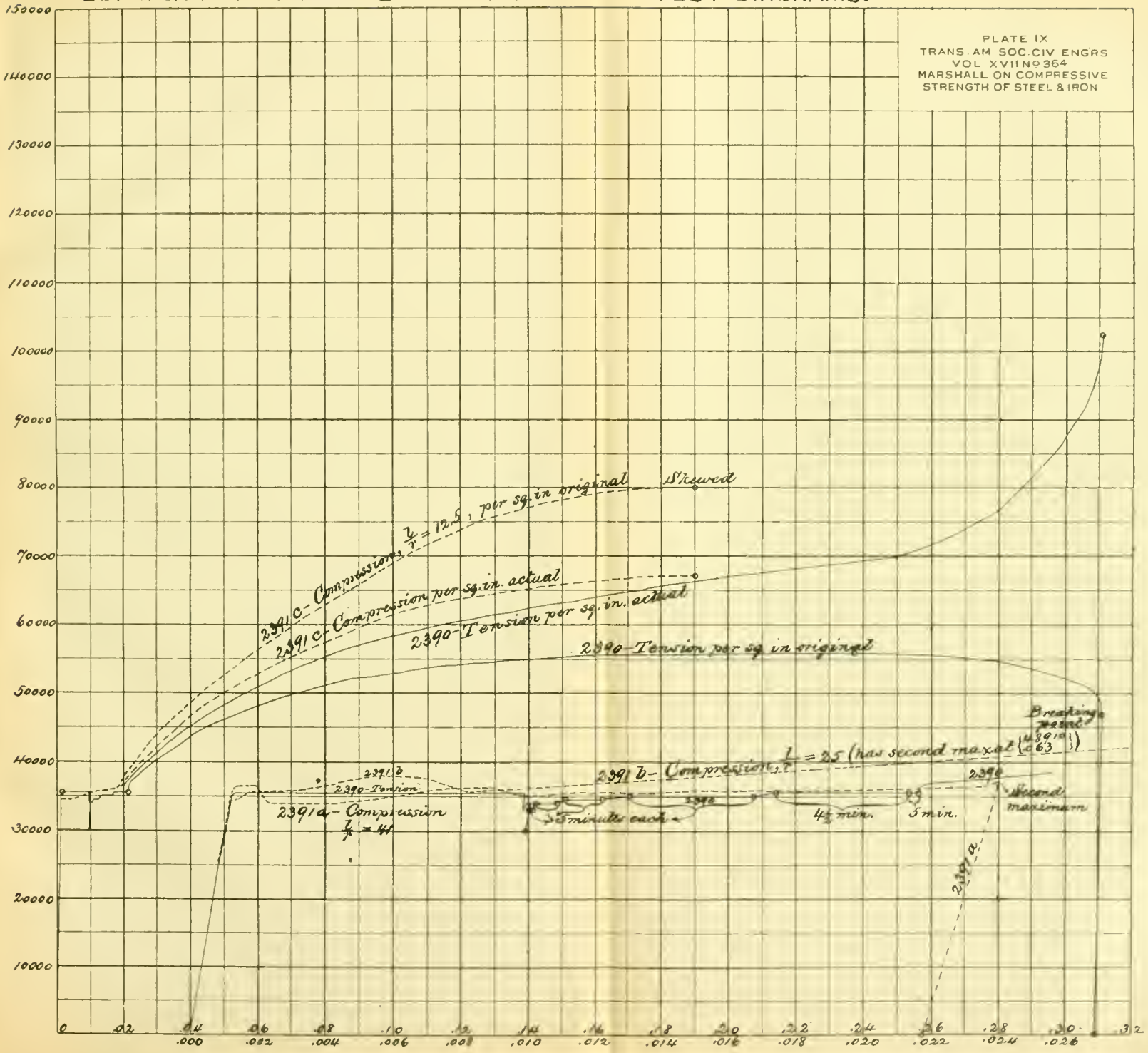



ONE INCH ROUND SPRING STEEL.
TEST DIAGRAMS.
ONE INCH ROUND 100000 LB. STEEL.
PLATE VII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVII No. 364.
MARSHALL ON COMPRESSIVE
STRENGTH OF STEEL & IRON.
Skewed
Skewed. (ends resquared)
Breaking point
547 - Compression, l/r = 12, per sq. in. original
548 - Tension per sq. in. actual
original
548 - Tension
549 - Compression l/r = 48
Maximum
547
549
Skewed
Skewed (ends resquared)
466 - Compression, l/r = 12, per sq. in. original
466 - Compression, per sq. in. actual
484 - Tension, per sq. in. actual
484 - Tension, per sq. in. original
Breaking point
484 - Tension
476 - Compression, l/r = 48
Second maximum

ONE INCH ROUND IRON, MADE FROM SCRAP. TEST DIAGRAMS.



SEVEN EIGHTHS INCH ROUND STAYBOLT IRON. TEST DIAGRAMS.



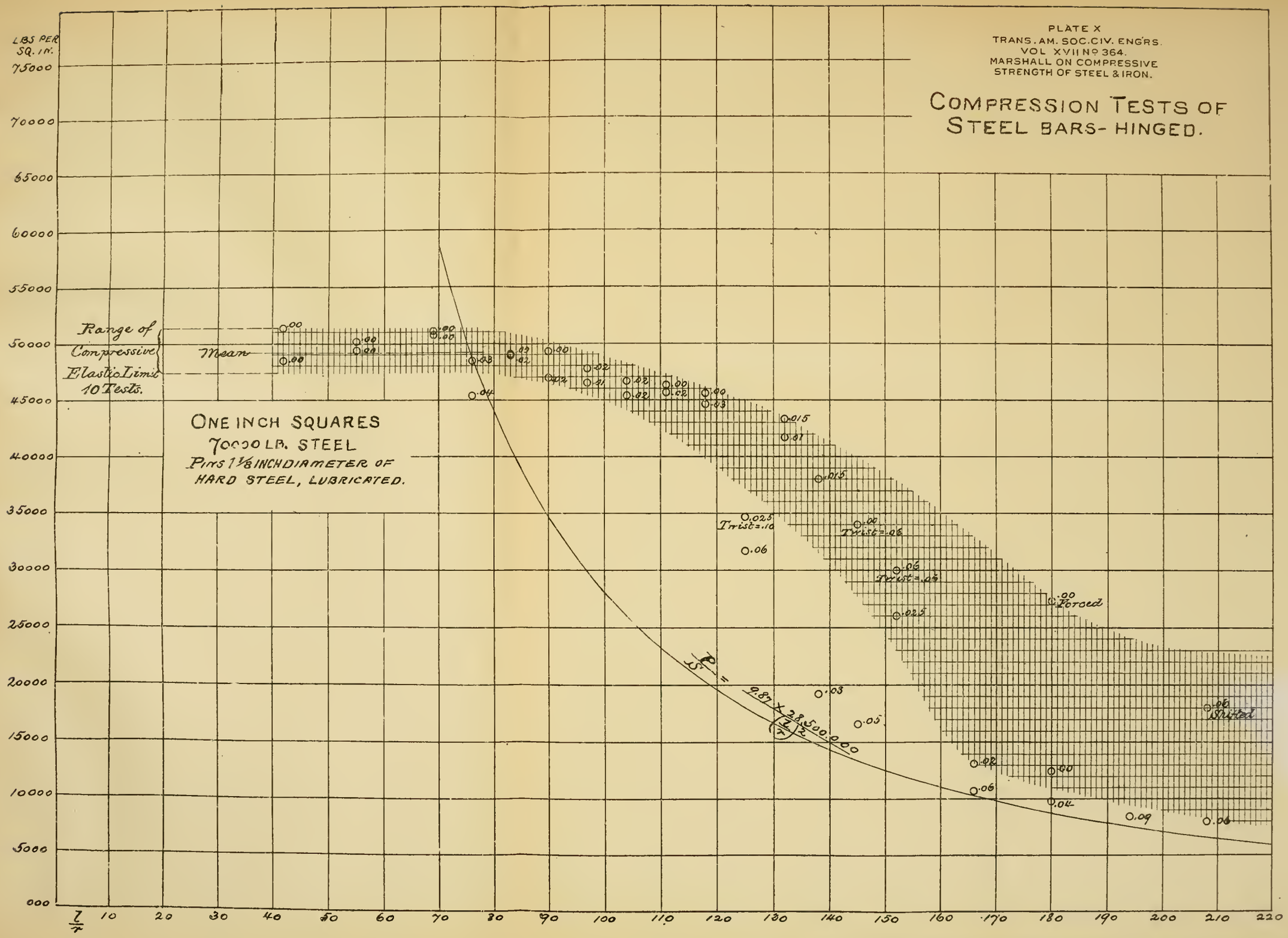


PLATE X
TRANS. AM. SOC. CIV. ENG'RS.
VOL XVII No 364.
MARSHALL ON COMPRESSIVE
STRENGTH OF STEEL & IRON.
COMPRESSION TESTS OF
STEEL BARS- HINGED.
LBS PER SQ. IN.
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
000
Range of Compressive Elastic Limit 10 Tests.
Mean
ONE INCH SQUARES
70000 LB. STEEL
PINS 1 1/8 INCH DIAMETER OF
HARD STEEL, LUBRICATED.
Twist=.10
Twist=.06
Twist=.04
Forced
Shifted
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220

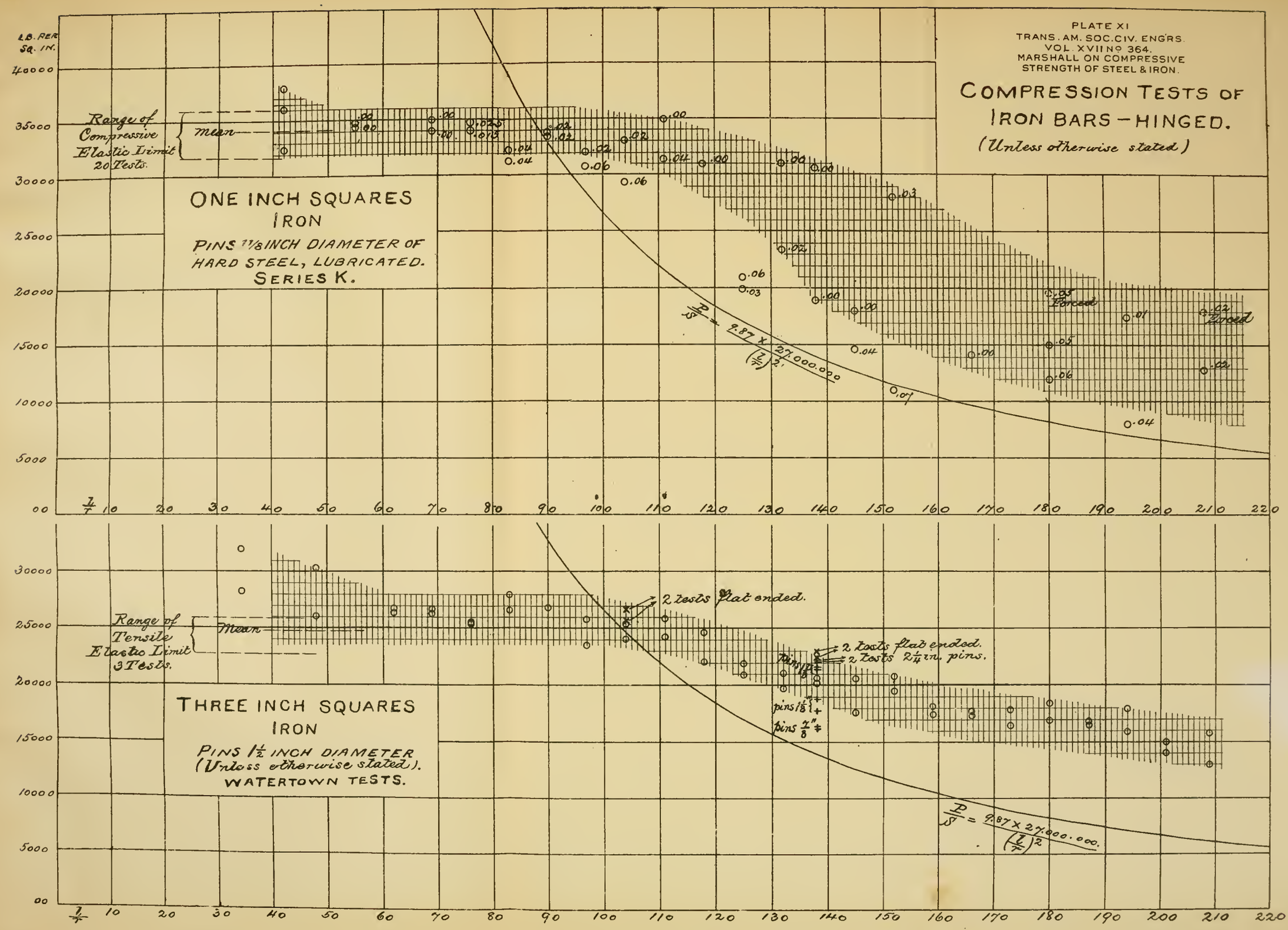
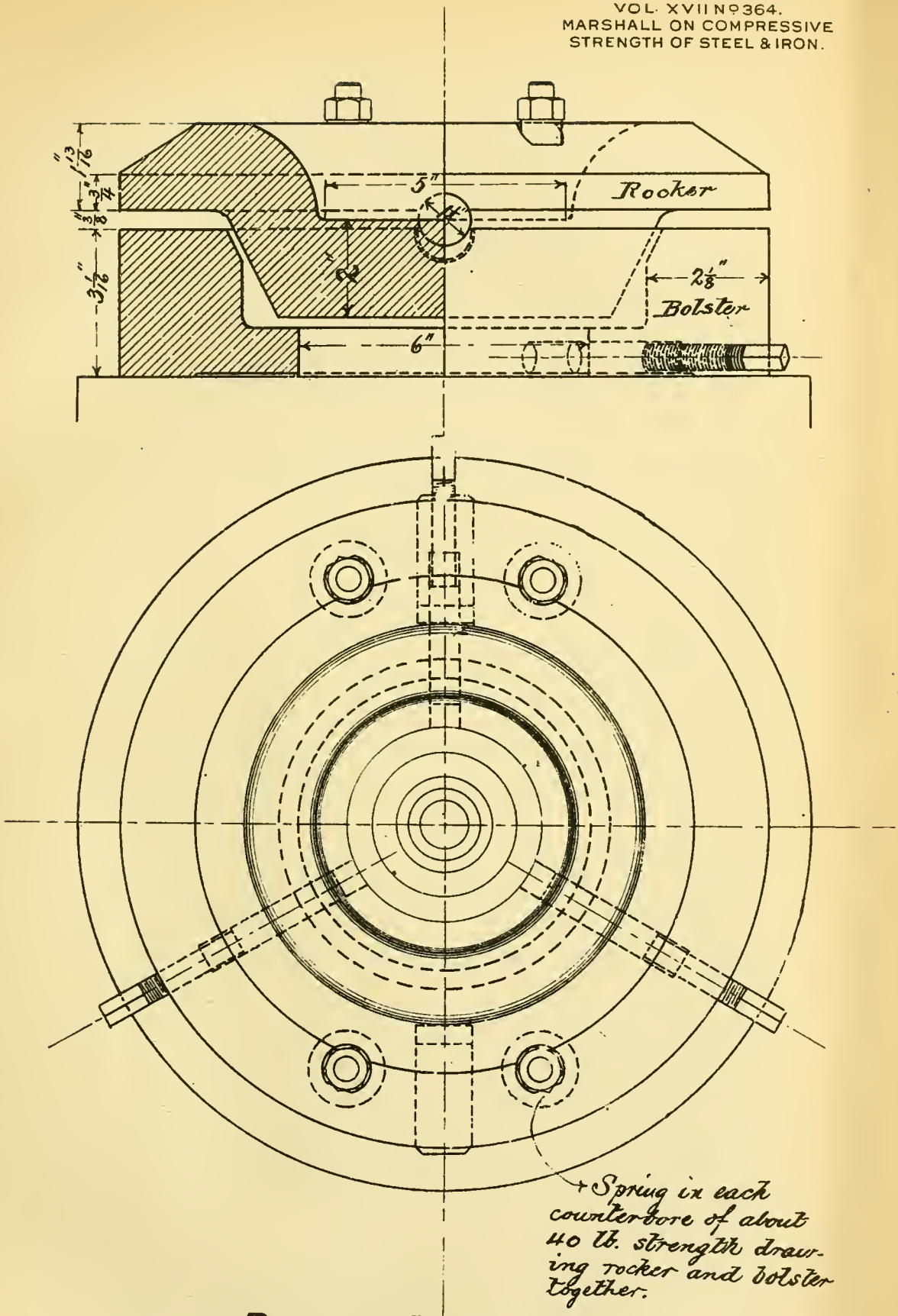


PLATE XI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XVII No 364.
MARSHALL ON COMPRESSIVE
STRENGTH OF STEEL & IRON.
COMPRESSION TESTS OF
IRON BARS — HINGED.
(Unless otherwise stated)
LB. PER SQ. IN.
Range of Compressive Elastic Limit 20 Tests.
mean
ONE INCH SQUARES
IRON
PINS 1 1/8 INCH DIAMETER OF
HARD STEEL, LUBRICATED.
SERIES K.
P/S = 9.87 X 27,000,000 / (l/r)²
Forced
Range of Tensile Elastic Limit 3 Tests.
mean
2 tests flat ended.
2 tests flat ended.
2 tests 2 1/4 in. pins.
pins 1 1/8"
pins 1 1/8"
pins 7/8"
THREE INCH SQUARES
IRON
PINS 1 1/2 INCH DIAMETER
(Unless otherwise stated).
WATERTOWN TESTS.
P/S = 9.87 X 27,000,000 / (l/r)²
l/r




ROCKER BEARINGS FOR

HINGED END COMPRESSION TESTS.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

365.

(Vol. XVII.—September, 1887.)

# THE KENTUCKY AND INDIANA BRIDGE.

By MACE MOULTON, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JULY 1ST, 1887.

Believing that the above-named structure is of sufficient magnitude to merit description, and the record of the progress and development of the details of the work of sufficient value to be of interest to the profession, the writer submits the following as a correct and tolerably full account of the construction.

The Directors of the new railroad being built between Louisville and St. Louis, *via* Evansville and New Albany, Ind., which was nearing completion in January, 1881, had, at an early stage of the enterprise; considered it expedient, and necessary to the interests of the road, to have a new bridge built crossing the Ohio River at Louisville, giving them a shorter and an independent entrance into the city. They therefore obtained charters in Indiana and Kentucky for such a structure, the former on March 2d, 1875, and the latter on April 1st, 1880.

These gentlemen, with others, formed a stock company, to be known as the Kentucky and Indiana Bridge Company, incorporated under the laws of Kentucky on February 1st, 1881, and of Indiana, March 7th, 1881. The capital stock was, after carefully detailed estimates, placed at $1 500 000.

The general location of the structure having been determined upon as connecting the lower part of the city of Louisville, known as Portland, and the upper part of the city of New Albany, Ind., the Stockholders and Directors appointed Mr. John MacLeod, M. Am. Soc. C. E., of Louisville, as Chief Engineer, and Mr. C. Shaler Smith, M. Am. Soc. C. E., of St. Louis, as Consulting Engineer.

The preliminary surveys of the river were commenced April 10th, 1881, and finished in about two months. These covered shore lines for two miles, sounding and boring, and current observations at three stages of water. The locations of the usual courses of vessels of different kinds crossing the proposed line were determined with the assistance of the chief falls pilot, in order to accommodate the river interests as much as possible in the disposition of spans and location of piers.

A map embodying all the results of surveys, together with the arragement of spans and elevation of grade finally determined upon, was made and forwarded to the War Department for approval.

From gauge records of the Government canal, as to stages of water for different seasons for several previous years, was plotted a hydrograph, which, together with the results of later observations up to completion of bridge is shown by Plate XV. The map of location sent to Washington is shown by Plate XIV.

The original idea of the bridge comprehended a structure capable of accommodating railway and highway traffic, to compete for the former with the upper bridge built by the Louisville Bridge Company—Mr. Albert Fink, Past President Am. Soc. C. E., Chief Engineer ; Louisville Bridge and Iron Company, contractors—now principally owned and operated by the Pennsylvania Railroad; and to supply facilities for the latter which had been but indifferently afforded by the existing ferry.

The site selected is at the narrowest point of the river, and admirably suited to accommodate both cities at its termini, and the railroads expected to run over it.

At this point the difference between extreme low and high water is greater than at any station on the river between Pittsburgh and Paducah, both included.

This difference, according to the data afforded by the great rise of 1832, is 67½ feet, and the Act of Congress providing for the construction of bridges over the Ohio requires that the lowest point of any structure spanning the channels of the river at this point shall be 40 feet above ex-

treme high water, thus necessitating the bottom chord center line to be about one hundred and nine feet above lowest low water.

The river is divided at the line of bridge into two chutes by an island, the Indiana chute being the wide and shallow, or high-water channel, and the Kentucky chute the narrow and deep channel used at low stages of water.

The current in the latter runs at all times with much force, which, together with the fact that very low water forces all traffic through this channel, made it advisable to avoid the use of false-work in erecting the span crossing it.

Close to the Indiana side of the island the bulk of the coal traffic passes, and the lower end of the island is used for a meeting point for the purpose of shifting tows of coal boats.

These facts made a long span necessary at this point, and a system of cantilevers suggested itself as best accommodating the circumstances of the location.

Taking all these governing points into consideration, the arrangement of spans as finally adopted was as shown by Plate XVI. This gives a total length between centers of end piers 2 453 feet. In this length the whole structure between Piers 4 and 9 is a continuous cantilever system 1 843 feet in length, the longest yet constructed, as far as known to the writer.

The design upon which the first estimates were based is shown in skeleton profile in Plate XVI.

The details of the substructure, as specified in contracts, comprehended the excavation of foundations for piers to bed-rock, which consists of limestone overlaid by slate of variable thickness. The stone to be used in the piers to be Bedford oolitic limestone, with courses up to 18 feet above low water, 37 inches in thickness; stretchers not over 8 feet 6 inches long by 3 feet wide; and headers not less than 6 feet long by 3 feet wide. Above this level the courses to vary, with minimum thickness of 2 feet 1 inch. The specifications for stone and for setting the same were rigid, insuring first-class work.

The quarries at Bedford are large and well equipped with channeling machines, and could get out the stone for the thick courses at a cost less than for smaller stone. Hence the bids showed that there was no additional cost, incident to the increased size of stone, in obtaining the material, and it was believed that time might be gained in setting

the stone in the wall and an improvement made in the solidity and appearance of the finished work. Piers 1 and 9 were to be of wrought-iron cylinders filled with concrete.

The superstructure was to be in general outline as already mentioned.

The cantilever system to be of steel in the trusses and iron in floor, and the remaining spans entirely of iron. The floor was to be riveted between trusses.

Unit strains in steel as follows:

| | | |
|---|---|---|
| Tension in eye-bars | 13 500 | pounds per square inch. |
| Riveted chords in 20-foot lengths | 14 000 | " " |
| Posts, 1.4 times their value in iron. | | |
| Iron parts: | | |
| Eye-bars | 10 000 | " " |
| Posts by adaptation of Rankine's formula. | | |
| Shape-iron tension, net section | 10 000 | " " |
| " compression, gross section | 8 000 | " " |

Estimates were carefully prepared of quantities and work to be done, and on September 16th, 1881, the Stockholders and Directors of the Bridge Company conferred on the President the authority to call for bids on the work.

The successful bidders were as follows:

Foundations: Peter Scully, St. Louis, Mo.
Stone: Hinsdale Doyle Granite Company, Bedford. Ind.
Freight on stone: Louisville, New Albany and Chicago Railway Company.
Laying masonry: McAtee, Flannery & Cassilly, Louisville, Ky.
Superstructure: Edge Moor Iron Company, Wilmington, Del.

The estimated cost of bridge, based on these bids, was as follows:

| | |
|---|---|
| Pier 1, cylinder pier | $12 237 87 |
| " 9, " | 20 217 23 |
| Piers 2 to 8 inclusive, masonry piers | 263 113 00 |
| Total substructure | $295 568 10 |
| " superstructure | 719 053 00 |
| Total | $1 014 621 10 |

Contracts were at once signed with the contractors for the substructure.

Ground was first broken in foundation for Pier 2 October 10th. The excavation was carried to bed-rock and cleared off in three days, the foundation being dry. On October 29th the caisson for Pier 3 was pumped dry and excavation commenced.

Bed-rock was reached at a depth of three feet on November 4th, but the foundation was not completed, as the river began to rise very rapidly, and in a short time the caisson was overflowed and work stopped.

On November 9th the first stone was set at Pier 2, but little was done, however, as the rise in the river soon covered the work.

Much valuable time was lost by the masonry contractors through mismanagement, and the plant being put up by them was inadequate to handle the very large stone in the lower courses. Hence the work was partially taken into the hands of the Bridge Company, and finally the contractors gave it up, and the contract was annulled by mutual consent on December 5th.

During the winter and spring nothing of consequence was done in the river. Stone was received from quarries and unloaded.

The first great rise in the river for many years occurred in February, culminating on the 22d at an elevation of 61 feet above low water.

The spring passed and the water did not subside to a point low enough to reclaim the foundations at Piers 2 and 3, and the only activity about the site was the receiving and unloading of stone.

Complications arose when the signing of traffic contracts with the Louisville, Evansville and St. Louis Railway, and the Louisville, New Albany and Chicago Railway was brought up, the directory of these roads having changed during the interval. Negotiations between the Bridge Company and the railroads not progressing satisfactorily, work was entirely suspended May 15th, 1882, and the contractors so notified.

The work thus stopped was left as follows: Foundations 2 and 3 excavated, but lost in high water; a large amount of stone in yards ready for laying; the entire plant of masonry contractors in possession of Bridge Company by purchase; and the sheets of cylinder iron for Piers 1 and 9 on the ground.

Nothing further was done towards construction until October 10th, 1883, when work was recommenced. Foundation for Pier 2 was reclaimed and stone-setting started. A new caisson for Pier 3 was put in place, being sunk as building progressed until bed-rock was reached. The river rose and drove them out just as it was anchored down. Some excavation was done on shore for Pier 1, and some riveting up of sheets of cylinders.

On December 10th, 1883, the contract for foundations and setting stone was let to Messrs. Bruce & Alexander, of Indiana, who immediately took charge of the plant and went on with the excavation at Pier 1, and commenced sinking the cylinders.

Work was continued with varying success until the great rise of the river in February, 1884, which reached its height on February 16th, at an elevation of 70.6 feet above low water, or four feet higher than ever before known. No damage was done by the rise at the bridge site.

The water subsided rapidly, and on April 14th excavation was recommenced at Pier 1, and from that time the work was prosecuted vigorously whenever the river permitted, until the last stone was set on the last pier July 6th, 1885.

The following is a brief summary of the progress on the various piers in the order of numbers, commencing at the Indiana shore, without reference to their chronological order, the work being necessarily intermittent on any particular pier, inasmuch as it was attempted to bring each in turn above ordinary stage of water as soon as possible after the foundation was obtained, and then proceed to others which were in a less forward state.

Cylinder Piers 1 and 9.—Each pier is composed of two cylinders, 7 feet diameter under coping, with a uniform batter of $\frac{1}{2}$ inch to a foot. The cylinders are formed of wrought-iron sheets $\frac{3}{8}$ and $\frac{1}{2}$-inch thick.

The lower or cutting edges are made of segmental castings, 9 inches wide at top, and tapered to an edge at bottom. On the circular shelf thus afforded just inside the shell is laid a 13-inch brick wall extending to the top of cylinder.

The cylinders are completely filled to within 15 inches of the top with hydraulic cement-concrete thoroughly rammed. The proportions of this concrete are, 1 part cement, 2 parts sand, and 6 of broken stone which will pass through a $2\frac{1}{2}$-inch ring. This concrete was put in at Pier 9 with the clam-shell dredge used in excavation, and with funnel-chutes around the edges. Portland cement was used in the concrete for the lower 10 feet of cylinders.

On top of the concrete was set a course of stone projecting two inches out of the cylinders, and on this was set the coping course. By this arrangement no vertical pressure is allowed to come on the wrought-iron shell.

Pier 1.—Work commenced in earnest December 10th, 1883, and was prosecuted until the 25th, when the cylinders struck rock. They were at that time 13 feet high. Nothing further was done till April 14th, 1884, when an order was issued to take them out and set them closer together to accommodate new design of bridge. After hoisting out the cylinders the slate was excavated until the bed-rock was reached and then they were lowered into place, and on May 1st the first cylinder was leveled and brick-laying and concreting commenced. This was continued until May 29th. Nothing more was done until July 9th, when work was carried on ten days and again stopped.

On August 6th the pier was again taken in hand, and on the 23d concreting was finished and the pier left. The coping was set on February 23d, 1885, thus finishing the work on Pier 1.

Pier 9.—Excavation began September 22d, 1884. Work on this pier was carried on steadily, with various discouraging mishaps due partly to the nature of the soil through which the cylinders were sunk, and partly to the insufficiency of the contractor's plant. Excavation was done by aid of pumps in the interior of cylinders. When the water could no longer be kept down a clam-shell dredge was used.

The brick-wall linings were built up continuously as the sinking progressed, giving a sufficient load, in addition to the weight of cylinders, to cause them to sink rapidly when excavation progressed favorably.

The earth around the cylinders was caving in most of the time, due to the running in of the sand on the lower level on the shore side. This caused them to slide and tip slightly, but whenever this occurred it was corrected by jacking the cylinders over during sinking.

The cutting edges struck rock May 12th, and after cleaning off the surface the concreting was rapidly pushed and the cylinders filled, and last stone in coping was set on July 6th, 1885.

River Piers.—These average 9 feet wide by 36 feet long on top under coping, and 106 to 120 feet high, with a batter all around of one-half inch to the foot.

All these are masonry piers resting on rock. Although without ornamental work of any kind, they present an appearance of solidity which, considering their height, is seldom exceeded.

In obtaining the foundations several methods were pursued. On the Indiana side the surface of the bottom was bare, and the strata of limestone on which it was designed to rest the piers dips downward towards the Kentucky side at an angle of about seven degrees with the horizontal.

The limestone is overlaid with thin sheets of slate, until Pier 7 is reached, where it runs out. This slate was easily blasted out, and the surface of the limestone immediately under it proved for a thickness of from ½ to 2 inches nearly as hard as flint.

The sand and gravel overlying the slate commenced at Pier 3 with a thin covering, and increased in depth towards Kentucky, until at Pier 8, 15 feet was encountered.

Pier 2.—As already stated, the foundation for this pier was obtained dry at a low stage of water, nothing but a puddle wall being used at any time in setting the lower courses of stone.

Work on the wall was commenced in earnest May 1st, 1884, and prosecuted vigorously, with few interruptions, until July 11th, when it was ready for coping. The first 40 feet was built with derrick on shore, and then as water rose, use was made of derrick boats for 25 feet more. The remainder was carried on by derrick set on the pier itself, and jacked up as the work progressed.

On July 9th, 1885, the coping course was laid, finishing the pier.

Pier 3.—At this pier the foundation was, as previouly mentioned, nearly finished in 1881, when the rising water filled the caisson.

This caisson was an octagonal box, of 2-inch vertical pine plank thoroughly caulked. Inside rings of 10 by 12-inch timber were placed every four feet in height, braced by oak timber bolted on top horizontally across the corners. The holding-down key-bolts passed through these braces into the rock.

This caisson was entirely lost in the high water of 1882.

A new one was begun October 23d, 1883, in plan also octagonal, of 2-inch plank, but laid flat, with hot pitch between the layers. The lower four feet was of 12-inch-wide plank, next four feet of 10-inch plank, next four feet of 8-inch, and next 6-inch plank, each layer being securely spiked to the one next below. This was jacked down as fast as built, and when finally in place on bottom was bolted securely to rock on outside.

As with the other box, the bottom was no sooner reached and all made secure, than the river rose and drove them out. Nothing more was seen of the caisson until after the great rise of February, 1884. On May 27th it began to show again, and on July 22d the water had subsided enough to work, and pumping was commenced and mud excavated. The next day the first stone was set in this pier.

This work was carried on about two-thirds of the time until November, when it was stopped a few courses below the coping.

During the first week in June, 1885, the pier was finished by setting the last coping stone June 8th.

PIER 4.—This foundation, commenced June 26th, 1884, was put in with a coffer-dam, built as follows:

Two rectangular frames of 12 by 12-inch timber were made, the outside frame about ten feet larger all around than the inside one. These were braced and bolted together in the same horizontal plane. Two other frames were then made, and braced together about four feet apart, of such size that the outside frame was three feet smaller all around than the outside frame of the former pair, and the inside frame three feet larger than the other inside frame.

The last pair was then put in place, ten feet vertically above the first pair and 1½-inch plank sheet piling driven close inside the frames, the space filled with puddle forming a wall ten feet thick at bottom and four feet thick at top. The whole rested on gravel.

No success in pumping dry was obtained until the inside sheet piles were driven to rock.

July 1st high water claimed the dam, but on the 15th day of the same month gave it up, and pumping-out commenced, and excavation was completed and first stone set July 19th. Three feet depth of gravel was encountered, and three feet of slate was blasted out before reaching bed-rock.

Masonry was laid continuously, with a few interruptions, during July and August, and again during November and the first half of December, when the pier was nearly completed.

During the first week in April, 1885, this pier was again taken up, and the last stone set April 9th.

PIER 5.—This foundation was obtained by means of a timber-dam with puddle outside.

The lower four feet of dam was composed of courses of 12 by 12-inch timbers, securely connected with each other by vertical bolts, and the upper section of six feet in similar courses of 10 by 10-inch timber, all being flush on the outside.

July 18th the box was floated into position. Sheet piling was then put in on outside and driven to rock, and puddle placed all around outside of piling. This proved a very satisfactory dam.

August 6th the water rose enough to stop work for ten days, at the end of which time pumping was commenced.

As above stated, puddle was found necessary, and after putting this around the outside on August 17th, excavation proceeded successfully to completion on August 23d, when the first stone was set.

Eighteen inches of gravel and three feet of slate were encountered in this foundation.

Stone-setting progressed fairly well, work being carried on about one-half of the time, until November 5th, when it was stopped for the winter.

On March 13th, 1885, work was again commenced, and continued until April 21st, when the last coping stone was set.

PIER 6.—The foundation for this pier was obtained by the use of an open box framed at site. This was commenced July 31st, 1884, and made 8 feet high, of vertical 1½-inch plank with frames of 12 by 12-inch timber every four feet, to which the planks were spiked and then caulked.

When this was sunk another four-foot section was added, and August 6th work was stopped by the rise of water. During this interval of rest, borings were made to determine how much gravel and rock were

to be encountered. The former proved to be 12 feet in depth and the slate 4 feet.

Excavation commenced August 28th and was prosecuted successfully until September 11th, when it was completed and the first stone set.

Laying stone was pushed vigorously until October 15th, when work was suspended for a month and again prosecuted from November 14th to December 13th. From this time the masonry was carried up in a desultory manner a few days at a time until April 2d, 1885, when the last coping stone was set, completing the pier.

Pier 7.—This foundation was taken in hand by the Bridge Company to insure its completion during the current season.

Two rows of piles were driven 7 feet apart, and 7 feet center to center in the rows. The inside rectangle thus formed was 20 feet wider and 25 feet longer than the proposed dam to be placed inside.

Between the rows were placed wales 2 to 3 feet apart vertically, 6 by 12-inch timber on outside of the inside rectangle of piles and 6 by 10-inch on the inside of outside rectangle.

Three-inch oak sheet piling was then driven close to the wale pieces of the inside row 4 feet into the gravel, and 2 inch planks on the outside wales were driven just enough to penetrate the gravel. This, when completed, left a space between sheet piling 4 feet inside all around, which was then filled with puddle.

The caisson was an open box of vertical 2-inch plank spiked on rectangular frames of 12 by 12-inch timbers placed four feet apart. The bottom frame was in two courses of 12-inch and the others single. The box was first built 8 feet high, and as sinking progressed another section of 7 feet was added.

The space between puddle walls and caisson was 10 feet wide all around, except at the down stream end, which was made 15 feet.

On August 29th, 1884, soundings having been completed, pile-driving commenced, and on September 19th puddle dam was completed. The next day the caisson was put together in place and excavation commenced and pushed night and day until October 8th, when bed-rock was cleared off.

Considerable time was lost from breakage of pumps, but no further delays of consequence occurred, and the ease with which the foundation was obtained confirmed the judgment used in adopting the method followed for this foundation. About fourteen feet of gravel and sand were excavated, and overlying this was a layer of 4 feet of loose rock which had washed down from work done on canal above in 1869. No slate was found.

Masonry was laid continuously from the time the bed-rock was reached for a month, when work was virtually suspended for the winter, the wall extending above ordinary high water.

On April 14th, 1885, work was again started, and continued until May 28th, when the last stone was set.

Pier 8.—The foundation for this pier was put in by the Bridge Company for the same reason as in case of Pier 7.

Owing to expected high water a pneumatic caisson was used to obtain this foundation.

Soundings were taken and borings made with a diamond drill during the first of September, 1884. In October guide piles were driven near the site for platforms.

On November 14th the caisson was launched on the Kentucky shore, and on the 16th it was in place. This caisson was built with vertical

sides 3 feet thick, of 12-inch timbers placed lengthwise. The working chamber was 7 feet high from cutting edges to roof. The latter was 5 feet thick in 12-inch layers; lower course lengthwise, next crosswise, the next two diagonal, and the top course lengthwise.

Posts were framed for open section above the roof, but as the stage of the water allowed work to be pushed without interruption, the masonry could be carried up faster than the caisson was sunk, and the sides were not needed.

On November 20th the bottom floor was knocked out and excavation and stone-setting commenced on the 22d. Sand-blowing was begun and excavation was carried on without serious interruption until rock was struck on the upper end December 4th. The rock was cleared off and the down-stream end of caisson blocked up carefully, and concreting commenced next day.

Laying stone was suspended on the 8th. The filling of the working chamber was completed December 24th, and the air shafts removed and their places filled.

About fifteen feet of gravel and sand were encountered in this foundation, but no slate.

The work, which had been carried on night and day during the sinking and concreting, was now stopped for two weeks until January 9th, 1885, when masonry was laid for a week, and also a week's work was put in during the early part of February, extreme cold weather preventing continuous stone-setting.

On April 23d this pier was again taken in hand, and work progressed continuously until June 6th, when the last stone was set.

## Superstructure.

In December, 1883, the final contract for superstructure was executed with Messrs. Charles Macdonald and Edward Hemberle, Members Am. Soc. C. E., on the basis of specifications given in Appendix, and design as shown in skeleton on Plate XVI. The general plans of details were to be furnished from the office of Mr. Hemberle, at Chicago, and the shop work was to be done at the shops of Messrs. Kellogg & Maurice, Members Am. Soc. C. E., at Athens, Pa.

On February 16th, 1885, the writer was directed by the Chief Engineer, Mr. John MacLeod, to assume charge of the work, with general supervision of drawings, bills of material, inspection of material at mills and shops, and of erection, with authority to represent him in all matters of detail.

In accordance with these instructions, the examination of designs was immediately entered upon at Chicago, and all stress sheets and drawings, both general and detail, were thoroughly checked over as regards conformity with specifications and skeleton designs upon which contract was based.

This work was carried on without interruption at Chicago, Buffalo

and Athens until the last pound of material was ordered. No unusual problems presented themselves in course of design, excepting those natural to the system of web members, and the series of cantilever-spans continued to such an unusual extent.

Among the noticeable features of design may be mentioned the use of steel pins 9 inches diameter at pier posts, the size of these posts themselves, which are 30 by 36 inches, using 36-inch steel plates 38 feet long, the largest steel plates ever rolled in American mills. Reference to the stress-sheet tables in the Appendix will also show some sizes which are not often encountered in ordinary bridges.

The entire cantilever system is of steel, with the exception of the floor, which is of iron.

The draw-span and the 240-foot span are composite, iron and steel in the trusses, with iron-floor systems. Early in the prosecution of the work, and as soon as Mr. Hemberle had finished the design of the cantilever system, he disposed of his interest in the contract to the Union Bridge Company, and later Mr. Macdonald also conveyed his interest to them, and from that time the entire contract was in their hands.

In placing the orders for the material, much discrimination was necessary to so scatter it that large quantities might be concentrated at shops simultaneously, so that no unnecessary stoppage might occur due to the delays of mills. In obtaining the iron required this was fairly successful, but in case of the steel prompt deliveries were not the rule. The causes which brought this about, however, were more due to want of duplication and the multiplicity of sizes incident to the design than to the mills.

The orders for steel angles, and bars for tension members, were placed with the Cambria Iron Company. Those for steel plates with Messrs. Carnegie Brothers & Co., who obtained their slabs from the Pennsylvania Steel Company, at Steelton, Pa.

The draw-span was sublet to the New Jersey Steel and Iron Company, who bought their steel billets and rolled their own angles, and had the steel plates from Carnegie's.

No steel-shape iron was used in the work.

By reference to the Appendix a complete record of tests made on the steel used in the structure may be seen, together with accompanying notes as to peculiarities observed, and deductions therefrom, made necessary in course of the prosecution of the work.

In the shops, construction was carried on as vigorously as shipments of material from the mills would allow, and the first shipment of 175 tons finished iron was made to bridge site in May, 1885.

During May and June false-work material was arriving at the river, and in July and August the false-work for 360-foot span was put in. This false-work was about one hundred feet high, and the water was at a very poor boating stage.

September 2d raising was commenced with iron-floor system. Up to this time 410 tons of iron had arrived.

During September 200 tons of steel for this span was received, and October 7–8th raising of the trusses was commenced.

November 12th pile-driving for false-work of Indiana 260-foot cantilever anchorage span was started, and on November 23d the erection of 360-foot span being completed, the 160-foot cantilever arm toward Kentucky was commenced and removal of false-work begun.

On December 3d 140 feet of cantilever was complete, as likewise was the false-work of 260-foot anchorage span. December 15th raising of Indiana cantilever, next 360-foot span, was commenced, also false-work for Kentucky 260-foot anchorage span. December 26th some missing parts required having arrived, erection of Indiana 260-foot span began.

January, 1885, opened cold, but work was pushed as much as possible, and on the 15th pits for sills of false-work of 240-foot Indiana shore span were started. All work in iron-raising was stopped by running ice, January 7th, on account of inability to transfer material barges. Progress was made, however, on both banks of river. On January 18th an iron-hull tug-boat was secured to handle barges, and work resumed on erection, and the 260-foot span was completed next day. The running out of 160-foot cantilever toward 360-foot span was immediately started, and as soon as its length reached 140 feet the other cantilever was resumed and both pushed out simultaneously.

On February 9th the junction was effected without difficulty, the device for adjustment shown by Plate XVII working very satisfactorily.

The Indiana traveler was removed and sent over to the Kentucky anchorage span, and the other traveler was run back to Kentucky cantilever first completed, to be ready to meet the other arm.

Meantime heavy ice had been running, and from the first had gorged somewhat above the false-work of the 260-foot span, giving much anxiety

lest it should be carried out; and that span having during the early ice stage no counterbalancing cantilever on its end, would, from its design, naturally collapse in event of losing the false-work. Immediate attempts were made to stiffen the trusses for such contingency, but the lower works withstood the jamb, being held down by the weight of iron-work.

As soon as safe, after enough weight of cantilever had been attached to the span, the removal of false-work was commenced at anchorage end. The two upper tiers for about one hundred feet had been sent down, when, January 24th, relieved of its superimposed weight, the lower tier for 100 feet rose with the ice and went out in a body and floated down the river erect as when in place. Finally, after floating in a vertical position about two miles, the mass collapsed into individual bents, in which shape the timber was caught some ten miles below.

This might have proved disastrous for the men, who were all over the work, but the slowness and apparent deliberation attending the movements of the false-work gave time for easy escape, which was effected without casualty.

February 12th, false-work for 240-foot shore-span was begun, and on the 17th erection of Kentucky anchorage-span commenced. The latter was completed on the 26th, and cantilever pointing toward Indiana was entered upon. This work was soon stopped, owing to delay in receipt of two eye-bars, and the other cantilever approaching this was projected out as far as possible, and on March 10th its traveler was removed.

On the same day the false-work for 240-foot span was completed and floor-raising was commenced. On the 19th work was again commenced on cantilever.

Rough and cold weather prevented continuous or effective work during the remainder of the month, and the river rose daily, until, on April 10th, it reached an elevation of 158 feet, or but 13 feet below the highest known high-water mark. On this day was celebrated the driving of the last pin of cantilever system, and next day the 240-foot span was swung. As fast as each span had been erected the wood-work of deck and highway, floors and fences, was commenced and pushed rapidly.

The false-work for draw-span between 260-foot Indiana anchorage-span, and 240-foot shore-span, was put in up and down stream, and was finished April 21. Erection then commenced, and was completed May 22d, and on the 26th the span was swung around into line, for the first

time allowing foot passage from shore to shore. June 8th the laying of railroad iron was commenced and finished on the 14th.

On June 21st the bridge was completed, and next day vehicles passed over for the first time. On July 15th the formal test of the structure was made by running over the spans with as nearly the specified loads as possible to obtain, these loads being so placed as to produce the conditions which the original calculations had shown would give the greatest stresses. Deflections were observed by taking sights with Y-levels placed on the piers on level rods held at critical points. These observations were subsequently compared with levels taken after all loads had been removed, and deflections thus ascertained. A record of results obtained will be presented to the Society in a future communication.

The following table gives results of a series of observations taken on the expansion and contraction of the openings in the slip joints marked *b*, Plate XVII, due to effect of temperature. The measurements on the iron were taken at point marked *x* in detail sketch, and for the wood at opening in highway floor just adjacent. The floor is so laid that the sun cannot at any time shine directly on the bottom chord where measurements were taken. A trap-door in the floor was left over every expansion joint to allow examination at any time. The measurements were all carefully taken by same observer, and the same thermometer was used throughout. To obtain the temperature of the iron, the thermometer was laid down, with the bulb resting on the member, out of the sun. The air temperature was also taken under the floor in the shade.

The question of proper amount to allow in slotting stringers and corbels of highway floors, where connected with the iron, was at first a vexatious one, as allowance had to be made for 520 feet. These observations were made by direction of the writer to determine whether the amount allowed was sufficient; and also to learn, if possible, if the movement of the iron under the wood was perfectly free.

As might be anticipated, the expansion shows itself at the openings in sudden slips, and from the results of observations there is little doubt that considereble bending sidewise sometimes occurs, due to friction somewhere before the movement takes place. As a matter of fact the writer has had the slip take place under the rule, being perfectly perceptible, and to the extent of $\frac{1}{32}$ inch.

The results show quite clearly that, owing to circumstances of position of sun with reference to the two trusses, the one expands more or less than the other, and it is also interesting to calculate the difference in deflection of the trusses under the varying circumstances, as may be easily done from the expansions and contractions, as shown in the table. Of course there are some variations from results to be anticipated, but the state of the weather would govern in some cases, and the unknown friction in some others.

The distance between fixed points is 360 feet plus 480 feet, or 840 feet, and the expansion takes place at both points *b*, sometimes more in one and sometimes more in the other, so that only the totals give the absolute expansion for the whole distance between fixed points.

## Table of Expansions and Contractions Due to Changes in Temperature.

| Time. | | East Truss, Ky. | | East Truss, Ind. | | West Truss, Ky. | | West Truss, Ind. | | Temp. | | Total East Truss | | Total West Truss. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Day, 1886. | Hour. | Iron. | Wood. | Iron. | Wood. | Iron. | Wood. | Iron. | Wood. | Air. | Iron. | Iron. | Wood. | Iron. | Wood. | Weather. |
| | | In. | In. | In. | In. | In. | In. | In. | In. | Deg | Deg | In. | In. | In. | In. | |
| Aug. 10. | 8 A.M. | 4 | 1 7/8 | 5 9/16 | 2 5/16 | 4 3/4 | 1 7/8 | 6 | 2 1/4 | 84 | 75 | 9 9/16 | 4 3/16 | 10 3/4 | 4 1/8 | Clear |
| " | 12 M. | 3 3/4 | 1 13/16 | 5 3/16 | 1 7/8 | 4 7/16 | 1 11/16 | 5 5/8 | 2 1/16 | 97 | 86 | 8 15/16 | 3 11/16 | 10 1/16 | 3 3/4 | " |
| " | 5 P.M. | 3 3/4 | 1 3/4 | 5 1/16 | 1 3/4 | 4 3/16 | 1 5/8 | 5 1/2 | 2 | 89 | 89 | 8 11/16 | 3 1/2 | 9 11/16 | 3 1/2 | " |
| Aug. 11. | 8 A.M. | 4 | 1 3/4 | 5 1/2 | 2 1/16 | 4 9/16 | 1 5/8 | 6 | 2 1/4 | 82 | 78 | 9 1/2 | 3 13/16 | 10 9/16 | 4 | " |
| " | 12 M. | 3 3/4 | 1 3/4 | 5 3/8 | 1 13/16 | 4 5/8 | 1 3/4 | 5 9/16 | 2 1/16 | 95 | 89 | 8 7/8 | 3 9/16 | 9 15/16 | 3 13/16 | " |
| " | 5 P.M. | 3 3/4 | 1 3/4 | 5 1/16 | 1 3/4 | 4 1/4 | 1 9/16 | 5 7/16 | 2 1/16 | 90 | 90 | 8 11/16 | 3 1/2 | 9 11/16 | 3 1/2 | " |
| Aug. 12. | 7.30 A.M. | 3 13/16 | 1 7/8 | 5 1/2 | 2 1/8 | 4 1/2 | 1 3/4 | 6 1/16 | 2 1/4 | 83 | 80 | 9 5/16 | 4 | 10 5/16 | 4 | Cloudy. |
| " | 12 M. | 3 13/16 | 1 3/4 | 5 3/8 | 1 15/16 | 4 1/2 | 1 3/4 | 5 13/16 | 2 1/8 | 90 | 86 | 9 5/16 | 3 11/16 | 10 5/16 | 3 7/8 | Clear. |
| " | 5.30 P.M. | 3 3/4 | 1 3/4 | 5 1/16 | 1 3/4 | 4 1/4 | 1 9/16 | 5 1/2 | 1 7/8 | 92 | 92 | 8 11/16 | 3 1/2 | 9 1/4 | 3 7/16 | " |
| Aug. 13. | 7.30 A.M. | 3 15/16 | 1 13/16 | 5 3/8 | 2 1/8 | 4 1/2 | 1 3/4 | 6 1/16 | 2 1/4 | 78 | 74 | 9 9/16 | 3 15/16 | 10 11/16 | 4 | Cloudy. |
| " | 12 M. | 4 | 1 3/4 | 5 3/4 | 2 1/16 | 4 9/16 | 1 3/4 | 5 13/16 | 2 1/16 | 86 | 82 | 9 3/8 | 3 13/16 | 10 3/8 | 3 13/16 | Fair. |
| " | 5.15 P.M | 3 3/4 | 1 3/4 | 5 1/16 | 1 3/4 | 4 5/16 | 1 3/4 | 5 1/2 | 1 7/8 | 91 | 90 | 8 13/16 | 3 1/2 | 9 13/16 | 3 1/2 | " |
| Aug. 14. | 8 A.M. | 3 15/16 | 1 3/4 | 5 3/4 | 1 7/8 | 4 1/2 | 1 3/4 | 5 7/8 | 2 1/16 | 84 | 82 | 8 7/16 | 3 3/4 | 10 3/8 | 3 11/16 | Cloudy. |
| " | 12 M. | 3 7/8 | 1 3/4 | 5 7/8 | 1 3/4 | 4 1/2 | 1 3/4 | 5 3/8 | 2 1/16 | 99 | 90 | 9 | 3 1/2 | 10 3/8 | 3 11/16 | Fair. |
| " | 5 P.M. | 3 3/4 | 1 3/4 | 5 1/4 | 1 3/4 | 4 3/4 | 1 1/2 | 5 1/2 | 2 1/16 | 90 | 90 | 9 | 3 1/2 | 10 1/4 | 3 9/16 | Cloudy. |
| Aug. 16. | 8 A.M. | 3 7/8 | 1 7/8 | 5 3/8 | 1 7/8 | 5 1/16 | 1 3/4 | 5 3/4 | 2 1/8 | 91 | 87 | 9 1/4 | 3 3/4 | 10 13/16 | 3 3/4 | " |
| " | 12 M. | 4 1/16 | 2 | 5 3/4 | 1 7/8 | 5 1/4 | 1 7/8 | 6 1/16 | 2 1/8 | 94 | 91 | 9 13/16 | 3 7/8 | 11 5/16 | 4 | Fair. |
| " | 6 P.M | 4 1/8 | 2 1/16 | 5 7/8 | 2 1/16 | 5 1/4 | 2 | 6 1/8 | 2 1/8 | 97 | 96 | 10 | 4 1/8 | 11 1/8 | 4 1/8 | Clear. |
| Aug. 17. | 8 A.M. | 3 3/4 | 1 3/4 | 5 3/8 | 2 | 4 7/16 | 1 1/2 | 5 7/8 | 2 1/16 | 88 | 82 | 9 1/8 | 3 3/4 | 10 5/16 | 3 9/16 | " |
| " | 2 P.M. | 4 1/16 | 1 7/8 | 5 7/8 | 2 1/16 | 5 1/16 | 1 5/8 | 5 3/4 | 2 1/8 | 99 | 94 | 9 15/16 | 3 15/16 | 10 13/16 | 4 | Cloudy. |
| " | 6 P.M | 3 3/4 | 1 3/4 | 5 3/8 | 1 7/8 | 4 3/8 | 1 1/2 | 5 3/8 | 1 7/8 | 89 | 88 | 9 1/4 | 3 1/2 | 10 | 3 1/4 | " |
| Aug. 18. | 8 A.M. | 4 | 1 3/4 | 5 7/8 | 1 15/16 | 5 1/2 | 1 3/4 | 6 1/8 | 2 1/8 | 74 | 70 | 9 7/8 | 3 11/16 | 11 5/8 | 3 7/8 | Rain. |
| " | 1 P M. | 4 1/16 | 1 7/8 | 5 3/8 | 2 1/8 | 4 3/4 | 1 3/4 | 6 1/16 | 2 1/8 | 76 | 72 | 9 1/2 | 4 | 10 13/16 | 3 7/8 | Cloudy. |
| " | 6 P.M. | 4 | 1 3/4 | 5 3/8 | 1 7/8 | 4 3/4 | 1 3/4 | 5 7/8 | 1 7/8 | 80 | 79 | 9 3/8 | 3 5/8 | 10 3/8 | 3 1/2 | Clear. |
| Aug. 19. | 7.30 A M. | 4 1/8 | 1 7/8 | 5 3/8 | 2 1/8 | 4 7/8 | 1 7/8 | 6 1/8 | 2 1/8 | 73 | 70 | 9 1/4 | 4 | 11 | 4 | " |
| " | 12 M. | 3 7/8 | 1 15/16 | 5 3/8 | 1 7/8 | 4 11/16 | 1 3/4 | 5 15/16 | 2 | 84 | 82 | 9 1/4 | 3 13/16 | 10 5/8 | 3 3/4 | " |
| " | 5.30 P.M. | 3 7/8 | 1 3/4 | 5 1/8 | 1 7/8 | 4 1/2 | 1 3/4 | 5 3/8 | 1 7/8 | 82 | 81 | 9 | 3 3/8 | 10 1/8 | 3 3/8 | " |
| Aug. 20. | 7.30 A M. | 4 1/16 | 1 7/8 | 5 7/16 | 2 1/8 | 4 13/16 | 1 13/16 | 6 1/8 | 2 3/16 | 74 | 72 | 9 1/2 | 4 | 10 15/16 | 4 | Fair. |
| " | 12 M. | 4 1/16 | 1 13/16 | 5 1/4 | 1 3/4 | 4 9/16 | 1 3/4 | 5 11/16 | 1 15/16 | 90 | 84 | 9 5/16 | 3 11/16 | 10 1/4 | 3 11/16 | Clear. |
| " | 5.30 P.M. | 3 7/8 | 1 3/4 | 5 1/8 | 1 3/4 | 4 1/4 | 1 3/4 | 5 3/4 | 1 7/8 | 88 | 84 | 8 7/8 | 3 1/2 | 10 | 3 1/4 | " |
| Aug. 21 | 7.30 A.M. | 4 7/8 | 1 3/4 | 5 1/2 | 2 1/8 | 4 1/8 | 1 3/4 | 5 7/8 | 2 1/8 | 76 | 74 | 10 1/8 | 3 3/4 | 10 | 3 1/4 | Cloudy. |
| Aug. 23. | 7.30 A.M. | 4 | 1 3/4 | 5 3/8 | 2 1/16 | 4 1/4 | 1 3/4 | 6 1/16 | 2 1/16 | 75 | 72 | 9 3/8 | 3 13/16 | 10 13/16 | 3 13/16 | " |
| " | 6 P.M. | 4 | 1 3/4 | 5 1/2 | 2 | 4 7/8 | 1 7/8 | 5 7/8 | 2 | 77 | 77 | 9 1/2 | 3 3/4 | 10 1/4 | 3 7/8 | " |
| Aug. 24. | 7.30 A.M. | 4 | 1 3/4 | 5 3/8 | 2 1/16 | 4 7/8 | 1 7/8 | 5 7/8 | 2 1/16 | 78 | 76 | 9 3/8 | 3 13/16 | 10 1/4 | 3 15/16 | Fair. |
| Aug. 25. | 8 A.M. | 3 7/8 | 1 3/4 | 5 1/2 | 2 | 3 7/8 | 1 3/4 | 6 | 2 1/16 | 80 | 78 | 9 3/8 | 3 3/4 | 9 7/8 | 3 13/16 | Clear. |
| " | 12 M. | 3 3/4 | 1 3/4 | 5 1/4 | 1 3/4 | 4 1/2 | 1 3/4 | 5 3/4 | 1 7/8 | 91 | 90 | 9 | 3 3/8 | 10 1/4 | 3 1/2 | " |
| Aug. 26. | 8 A.M. | 3 7/8 | 1 3/4 | 5 1/8 | 1 3/4 | 4 1/4 | 1 3/4 | 6 | 2 1/8 | 82 | 78 | 9 | 3 3/8 | 10 3/8 | 3 3/4 | Fair. |
| " | 5.30 P.M. | 3 3/8 | 1 3/8 | 5 1/16 | 1 3/4 | 4 1/4 | 1 1/2 | 5 7/8 | 1 3/4 | 92 | 90 | 8 11/16 | 3 3/8 | 10 1/8 | 3 3/4 | Cloudy. |
| Aug. 27. | 7.30 A.M. | 3 7/8 | 1 9/16 | 5 1/2 | 2 1/4 | 4 9/16 | 1 1/2 | 6 1/16 | 2 1/8 | 82 | 78 | 9 3/8 | 3 13/16 | 10 3/8 | 3 3/4 | Clear. |
| " | 12 M. | 3 7/8 | 1 3/4 | 5 1/2 | 1 3/4 | 4 1/2 | 1 3/4 | 5 7/8 | 1 7/8 | 99 | 92 | 9 | 3 3/8 | 10 3/8 | 3 1/2 | " |
| Aug. 28 | 7.30 A.M. | 3 7/8 | 1 3/4 | 5 1/4 | 1 7/8 | 4 1/2 | 1 3/4 | 6 | 2 1/16 | 82 | 80 | 9 1/8 | 3 1/2 | 10 3/8 | 3 11/16 | " |
| " | 12 M | 3 7/8 | 1 9/16 | 5 1/4 | 1 7/8 | 4 1/2 | 1 3/4 | 6 | 2 | 86 | 81 | 9 1/8 | 3 7/16 | 10 3/8 | 3 3/4 | Cloudy. |
| " | 5.30 P.M. | 3 3/4 | 1 7/16 | 5 1/4 | 1 3/4 | 4 1/2 | 1 3/4 | 5 7/8 | 1 7/8 | 80 | 80 | 8 7/8 | 3 3/16 | 10 3/8 | 3 1/2 | " |
| Aug. 30. | 8 A.M | 4 1/8 | 1 3/4 | 5 1/4 | 1 7/8 | 4 1/2 | 1 3/4 | 6 1/16 | 2 1/16 | 79 | 73 | 9 1/8 | 3 3/8 | 10 11/16 | 3 13/16 | Rain. |
| " | 12 M. | 3 7/8 | 1 5/8 | 5 9/16 | 2 | 4 3/8 | 1 3/8 | 6 | 2 | 78 | 74 | 9 7/16 | 3 3/8 | 10 3/8 | 3 3/8 | Cloudy. |
| " | 5 P.M. | 3 3/8 | 1 3/8 | 5 3/8 | 2 | 4 3/8 | 1 3/8 | 6 | 2 | 80 | 80 | 9 1/4 | 3 3/8 | 10 3/4 | 3 3/4 | Fair. |
| Aug. 31. | 7.30 A.M. | 4 1/8 | 1 3/4 | 5 3/8 | 2 1/8 | 4 1/2 | 1 3/4 | 6 3/16 | 2 1/8 | 70 | 66 | 9 3/8 | 3 7/8 | 11 1/16 | 3 7/8 | Clear. |
| " | 12 M. | 4 | 1 3/4 | 5 3/8 | 1 7/8 | 4 3/8 | 1 3/8 | 5 7/8 | 1 7/8 | 79 | 76 | 9 3/8 | 3 3/8 | 10 3/8 | 3 1/2 | " |
| " | 5 P.M. | 3 7/8 | 1 3/4 | 5 3/8 | 1 7/8 | 4 1/4 | 1 3/8 | 5 7/8 | 1 3/4 | 78 | 77 | 9 3/8 | 3 3/8 | 10 1/4 | 3 1/2 | " |
| Sept. 1. | 7.30 A.M. | 4 1/8 | 1 7/8 | 5 7/8 | 2 1/4 | 5 1/16 | 1 7/8 | 6 1/4 | 2 1/8 | 64 | 61 | 10 1/8 | 4 1/8 | 11 5/16 | 4 1/8 | " |
| " | 12 M. | 4 1/8 | 1 7/8 | 5 7/8 | 2 1/16 | 4 7/8 | 1 7/8 | 6 1/16 | 2 1/8 | 78 | 75 | 9 7/8 | 3 13/16 | 10 15/16 | 4 | Cloudy. |
| " | 5.30 P.M. | 4 1/16 | 1 7/8 | 5 3/4 | 1 7/8 | 4 1/4 | 1 3/4 | 6 | 1 7/8 | 76 | 74 | 9 13/16 | 3 3/4 | 10 1/4 | 3 3/4 | Clear. |
| Sept. 2. | 7.30 A.M. | 4 1/4 | 1 7/8 | 5 7/8 | 2 1/4 | 5 | 1 7/8 | 6 1/4 | 2 1/8 | 64 | 62 | 10 1/8 | 4 1/8 | 11 1/4 | 4 1/8 | " |
| " | 12 M. | 4 | 1 3/4 | 5 3/4 | 2 1/8 | 4 7/8 | 1 3/4 | 6 | 2 1/8 | 72 | 70 | 9 3/4 | 3 7/8 | 10 7/8 | 3 7/8 | " |
| Sept. 3. | 7.30 A.M. | 4 1/8 | 1 7/8 | 5 7/8 | 2 3/16 | 4 7/8 | 1 7/8 | 6 1/8 | 2 3/16 | 74 | 70 | 10 | 4 1/16 | 11 | 4 1/16 | " |
| " | 12 M. | 4 | 1 13/16 | 5 3/8 | 1 3/4 | 4 11/16 | 1 3/4 | 5 3/4 | 1 7/8 | 88 | 80 | 9 1/8 | 3 3/16 | 10 7/16 | 3 3/4 | " |
| Sept. 4. | 8 A.M. | 4 1/16 | 1 7/8 | 5 1/4 | 2 1/8 | 4 1/2 | 1 3/4 | 6 1/16 | 2 1/8 | 79 | 72 | 9 7/8 | 3 1/4 | 10 15/16 | 3 3/4 | " |
| " | 12 M. | 4 | 1 5/8 | 5 1/2 | 2 | 4 1/2 | 1 1/2 | 6 | 2 | 90 | 94 | 9 1/2 | 3 3/4 | 10 1/2 | 3 1/2 | " |
| " | 5.30 P.M. | 3 3/4 | 1 11/16 | 4 7/8 | 1 3/4 | 4 1/2 | 1 9/16 | 5 5/8 | 1 7/8 | 88 | 87 | 8 7/8 | 3 7/16 | 10 | 3 7/16 | " |
| Sept. 14. | 7.30 A.M | 4 9/16 | 1 15/16 | 6 3/16 | 2 3/16 | 5 1/8 | 2 | 6 3/8 | 2 3/8 | 65 | 59 | 10 3/4 | 4 1/8 | 11 1/2 | 4 1/8 | Fair. |

## Stress Sheet Tables to Accompany Stress Diagrams Plate XVIII. —Material, Steel.

160-foot Cantilever.

| Member. | Mark. | Required section. | Material used. |
|---|---|---|---|
| | | Sq. Inches. | |
| Top chord..... ..... | $U_0$ $U_2$ | 68.3 | 2 bars $7 \times 1\frac{7}{16}$. |
| | | | 2 " $7 \times 1\frac{9}{16}$. |
| | | | 2 " $7 \times 1\frac{7}{8}$. |
| " ..... | $U_2$ $U_4$ | 51.0 | 2 " $6 \times 2$. |
| | | | 2 " $6 \times 2\frac{1}{4}$. |
| " ..... | $U_4$ $U_6$ | 30.0 | 4 " $6 \times 1\frac{1}{4}$. |
| " ..... | $U_6$ $U_7$ | 17.25 | 2 " $6 \times 1\frac{7}{16}$. |
| | $U_7$ $U_8$ | 11.8 | 2 " $5 \times 1\frac{3}{16}$. |
| Bottom chord..... | $L_0$ $L_2$ | 76.0 | 2 20 $\times \frac{3}{4}$. |
| | | | 2 20 $\times \frac{11}{16}$. |
| | | | 4 L's $4 \times 4 \times 49$ lbs. |
| " ..... | $L_2$ $L_4$ | 57.0 | 2 20 $\times \frac{9}{16}$. |
| | | | 2 20 $\times \frac{3}{8}$. |
| | | | 4 L's $4 \times 4 \times 49$ lbs. |
| " ..... | $L_4$ $L_6$ | 38.2 | 2 20 $\times \frac{5}{8}$. |
| | | | 4 L's $4 \times 4 \times 33$ lbs. |
| " ..... | $L_6$ $L_7$ | 20.0 | 2 20 $\times \frac{5}{16}$. |
| | | | 4 L's $3 \times 3 \times 21$ lbs. |
| " ..... | $L_7$ $L_8$ | .... | 2 12 $\times \frac{5}{16}$. |
| | | | 4 L's $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$. |
| Web ..... | $U_0$ $M_1$ | 21.1 | 2 bars $7 \times 1\frac{1}{2}$. |
| " ..... | $M_1$ $L_2$ | 20.0 | 2 " $7 \times 1\frac{7}{16}$. |
| " ..... | $U_2$ $M_3$ | 19.0 | 2 " $6 \times 1\frac{3}{8}$. |
| " ..... | $M_3$ $L_4$ | 17.6 | 2 " $6 \times 1\frac{1}{2}$. |
| " ..... | $U_4$ $M_5$ | 20.06 | 2 " $6 \times 1\frac{11}{16}$. |
| " ..... | $M_5$ $L_6$ | 18.07 | 2 " $6 \times 1\frac{1}{2}$. |
| " ..... | $U_6$ $L_7$ | 12.08 | 2 " $5 \times 1\frac{3}{16}$. |
| " ..... | $U_7$ $L_7$ | 10.1 | 2 " $4\frac{1}{2} \times 1\frac{1}{8}$. |
| " ..... | $U_8$ $L_8$ | 18.8 | 4 " $4\frac{1}{2} \times 1\frac{1}{16}$. |
| | $L_0$ $M_1$ | 21.3 | 2 15 $\times \frac{7}{16}$. |
| | | | 4 L's $3 \times 3 \times 21.5$ lbs. |
| | $M_1$ $U_2$ | 15.2 | 4 15 $\times \frac{5}{16}$. |
| | | | 4 L's $3 \times 3 \times 18$ lbs. |
| " ..... | $L_2$ $M_3$ | 20.0 | 2 15 $\times \frac{3}{8}$. |
| | | | 4 L's $3 \times 3 \times 21.5$ lbs. |
| " ..... | $M_3$ $U_4$ | 15.14 | 2 15 $\times \frac{5}{16}$. |
| | | | 4 L's $3 \times 3 \times \frac{5}{16}$. |
| " ..... | $L_4$ $M_5$ | 15.7 | 2 14 $\times \frac{5}{16}$. |
| | | | 4 L's $2\frac{1}{2} \times 2\frac{1}{2} \times 17.5$ lbs. |
| " ..... | $M_5$ $U_6$ | 11.23 | 2 14 $\times \frac{5}{16}$. |
| | | | 4 L's $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$. |
| " ..... | $L_6$ $U_7$ | 12.1 | 2 12 $\times \frac{5}{16}$. |
| | | | 4 L's $2\frac{1}{2} \times 2\frac{1}{2} \times 15$ lbs. |
| " ..... | $L_7$ $U_8$ | 27.2 | 2 15 $\times \frac{1}{2}$. |
| | | | 4 L's $3 \times 3 \times 30.5$. |
| Suspenders ..... | $L_1$ $M_1$ | 6.2 | 2 bars $3\frac{1}{2} \times \frac{7}{8}$. |
| " ..... | $L_3$ $M_3$ | 6.2 | " |
| " ..... | $L_5$ $M_5$ | 6.2 | " |

360 FOOT SPAN.

| Member. | Mark. | Required section. | Material used. |
|---|---|---|---|
| Top chord ..... | $U_0$ $U_2$ | 71.93 | 1 24 × $\frac{3}{8}$.<br>2 18 × $\frac{3}{8}$.<br>2 L's 3 × 3 × $\frac{3}{8}$.<br>2 L's 4 × 4 × $\frac{1}{2}$.<br>2 bars 7 × $1\frac{1}{2}$.<br>2 " 7 × $1\frac{7}{16}$. |
| " ..... | $U_2$ $U_4$ | 66.1 | Same plates and angles.<br>2 bars 6 × $1\frac{1}{2}$.<br>2 " 6 × $1\frac{7}{16}$. |
| " ..... | $U_4$ $U_6$ | 63.32 | 1 24 × $\frac{1}{2}$.<br>2 18 × $\frac{3}{8}$<br>2 L's 3 × 3 × $\frac{3}{8}$.<br>2 L's 4 × 4 × 41 lbs. |
| " ..... | $U_6$ $U_8$ | 62.72 | 2 bars 6 × $1\frac{3}{8}$.<br>1 24 × $\frac{5}{8}$.<br>2 18 × 1.<br>2 L's 3 × 3 × $\frac{3}{8}$.<br>2 L's 4 × 4 × 37.6 lbs. |
| " ..... | $U_8$ $U_8$ | 62.4 | 1 24 × $\frac{5}{8}$.<br>2 18 × 1.<br>2 L's 3 × 3 × $\frac{3}{8}$. |
| Bottom chord..... | $L_0$ $L_2$ | 80.3 | 2 L's 4 × 4 × 36 lbs.<br>4 20 × $\frac{3}{4}$.<br>4 L's 4 × 4 × 50.7 lbs. |
| " ..... | $L_2$ $L_4$ | 73.3 | 2 20 × $\frac{3}{4}$.<br>2 20 × $\frac{5}{8}$. |
| " ..... | $L_4$ $L_6$ | 68.1 | 4 L's 4 × 4 × 45.8 lbs.<br>4 20 × $\frac{5}{8}$. |
| " ..... | $L_6$ $L_8$ | 66.89 | 4 L's 4 × 4 × 45.2 lbs.<br>4 20 × $\frac{5}{8}$. |
| " ..... | $L_8$ $L_9$ | 66.64 | 4 L's 4 × 4 × 42 lbs.<br>4 20 × $\frac{5}{8}$.<br>L's 4 × 4 × 42 lbs. |
| Web ..... | $U_0$ $M_1$ | 27 | 4 bars 6 × $1\frac{1}{8}$. |
| " ..... | $M_1$ $L_2$ | 25.6 | 2 " 5 × $1\frac{1}{8}$.<br>2 " 5 × $1\frac{5}{16}$. |
| " ..... | $U_2$ $M_3$ | 22.3 | 4 " 5 × $1\frac{1}{8}$. |
| " ..... | $M_3$ $L_4$ | 21 | 4 " 5 × $1\frac{1}{16}$. |
| " ..... | $U_4$ $M_5$ | 17.64 | 2 12 × $\frac{7}{16}$.<br>4 L's 3 × 3 × 26.5 lbs. |
| " ..... | $M_5$ $L_6$ | 17.0 | 2 12 × $\frac{7}{16}$.<br>4 L's 3 × 3 × 22.5 lbs. |
| " ..... | $U_6$ $M_7$ | 14.45 | 2 12 × $\frac{3}{8}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 17 lbs. |
| " ..... | $M_7$ $L_8$ | 14 | 2 12 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 17 lbs. |
| " ..... | $U_8$ $M_9$ | 14.7 | 2 12 × $\frac{3}{8}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_0$ $M_1$ | 33.7 | 2 16 × $\frac{5}{8}$.<br>4 L's 4 × 4 × 34.2 lbs. |
| " ..... | $M_1$ $U_2$ | 28.86 | 2 16 × $\frac{1}{2}$.<br>4 L's 4 × 4 × 32 lbs. |
| " ..... | $L_2$ $M_3$ | 27.3 | 2 15 × $\frac{9}{16}$.<br>4 L's 3 × 3 × 26 lbs. |
| " ..... | $M_3$ $U_4$ | 23.1 | 2 15 × $\frac{7}{16}$.<br>4 L's 3 × 3 × 25 lbs. |
| " ..... | $L_4$ $M_5$ | 21.7 | 2 15 × $\frac{7}{16}$.<br>4 L's 3 × 3 × 21.5 lbs. |
| " ..... | $M_5$ $U_6$ | 16.7 | 2 15 × $\frac{5}{16}$.<br>4 L's 3 × 3 × 18.3 lbs. |
| ..... | $L_6$ $M_7$ | 15.1 | 2 14 × $\frac{5}{16}$.<br>4 L's 3 × 3 × $\frac{5}{16}$. |
| " ..... | $M_7$ $U_8$ | 15.8 | 2 14 × $\frac{5}{16}$.<br>4 L's 3 × 3 × $\frac{5}{16}$. |
| " ..... | $L_8$ $M_9$ | 14.1 | 2 12 × $\frac{5}{16}$. |
| Suspenders ..... | $L_1$ $M_1$ | 6.2 | 4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 16.5 lbs.<br>2 bars $3\frac{1}{2}$ × $\frac{7}{8}$. |
| " ..... | $L_2$ $M_2$ | 6.2 | " |
| " ..... | Etc. | | " |

260-FOOT CANTILEVER.

| Member. | Mark. | Required section. | Material used. |
|---|---|---|---|
| | | Sq. inches | |
| Pier post | $L_0$ $U_0$ | 77.4 | 2 36 × $\frac{11}{16}$.<br>4 L's 4 × 6 × 70 lbs. |
| Top chord | $U_0$ $U_2$ | 74.0 | 2 bars 7 × $1\frac{1}{2}$.<br>2 " 7 × 2.<br>2 " 7 × $1\frac{25}{32}$. |
| " | $U_2$ $U_4$ | 68.4 | 2 " 7 × $1\frac{5}{8}$.<br>2 " 7 × $1\frac{9}{16}$.<br>2 " 7 × $1\frac{11}{16}$. |
| " | $U_4$ $U_6$ | 64.1 | 1 24 × $\frac{11}{16}$.<br>2 16 × 1.<br>2 L's 3 × 3 × $\frac{1}{2}$.<br>2 L's 4 × 4 × $\frac{11}{16}$. |
| " | $U_6$ $U_8$ | 61.7 | 1 24 × $\frac{3}{4}$.<br>2 16 × $\frac{7}{8}$.<br>2 L's 3 × 3 × $\frac{1}{2}$.<br>2 L's 4 × 4 × 51 lbs. |
| " | $U_8$ $U_9$ | 70.0 | 1 24 × $\frac{25}{32}$.<br>2 16 × $\frac{5}{8}$.<br>2 16 × $\frac{9}{16}$.<br>2 L's 3 × 3 × $\frac{1}{2}$.<br>2 L's 4 × 4 × 38.5 lbs. |
| " | $U_9$ $U_{10}$ | 59.2 | 1 24 × $\frac{3}{4}$<br>2 16 × $\frac{7}{8}$.<br>2 L's 3 × 3 × $\frac{1}{2}$.<br>2 L's 4 × 4 × 38.5 lbs. |
| " | $U_{10}$ $U_{11}$ | 48.75 | 1 24 × $\frac{3}{4}$.<br>2 16 × $\frac{5}{8}$.<br>2 L's 3 × 3 × $\frac{3}{8}$.<br>2 L's 4 × 4 × $\frac{7}{16}$. |
| " | $U_{11}$ $U_{12}$ | 35.1 | 1 24 × $\frac{7}{16}$.<br>2 16 × $\frac{7}{16}$.<br>2 L's 3 × 3 × $\frac{3}{8}$.<br>2 L's 4 × 4 × $\frac{7}{16}$. |
| Bottom chord | $L_0$ $L_2$ | 81.3 | 6 20 × $\frac{1}{2}$.<br>4 L's 4 × 4 × 53.3 lbs. |
| " | $L_2$ $L_4$ | 75.3 | 4 20 × $\frac{11}{16}$.<br>4 L's 4 × 4 × 50.8. |
| " | $L_4$ $L_6$ | 67.9 | 4 20 × $\frac{5}{8}$.<br>4 L's 4 × 4 × 44.8 lbs. |
| " | $L_6$ $L_8$ | 63.9 | 2 20 × $\frac{9}{16}$.<br>2 20 × $\frac{1}{2}$<br>4 L's 4 × 4 × 53.5 lbs. |
| " | $L_8$ $L_9$ | 66.0 | 4 20 × $\frac{9}{16}$.<br>4 L's 4 × 4 × 53 lbs. |
| " | $L_9$ $L_{10}$ | 53.8 | 2 20 × $\frac{13}{16}$<br>4 L's 4 × 4 × 53 lbs. |
| " | $L_{10}$ $L_{11}$ | 44.0 | 2 20 × $\frac{3}{4}$.<br>4 L's 4 × 4 × 35 lbs. |
| " | $L_{11}$ $L_{12}$ | 30.3 | 2 20 × $\frac{13}{32}$.<br>4 L's 4 × 4 × 35 lbs. |
| " | $L_{12}$ $L_{13}$ | 23.8 | 2 20 × $\frac{3}{8}$.<br>4 L's $3\frac{1}{2}$ × $3\frac{1}{2}$ × 22 lbs. |
| Web | $U_0$ $M_1$ | 20.47 | 2 bars 7 × $1\frac{15}{32}$. |
| " | $M_1$ $L_2$ | 18.85 | 2 " 7 × $1\frac{11}{32}$. |
| " | $U_2$ $M_3$ | 18.02 | 2 " 6 × $1\frac{1}{2}$. |
| " | $M_3$ $L_4$ | 16.63 | 2 " 6 × $1\frac{1}{8}$. |
| " | $U_4$ $M_5$ | 15.0 | 2 " 5 × $1\frac{1}{2}$. |
| " | $M_5$ $L_6$ | 14.0 | 2 " 5 × $1\frac{13}{32}$. |
| " | $U_6$ $M_7$ | 11.9 | 2 " 5 × $1\frac{3}{16}$. |
| " | $M_7$ $L_8$ | 11.3 | 2 " 5 × $1\frac{1}{8}$. |
| " | $U_8$ $L_9$ | 8.55 | 2 " 4 × $1\frac{1}{16}$. |
| " | $U_9$ $L_{10}$ | 8.93 | 2 10 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " | $U_{10}$ $L_{11}$ | 10.64 | 2 10 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " | $U_{11}$ $L_{12}$ | 15.54 | 2 13 × $\frac{5}{16}$.<br>4 L's 3 × 3 × 18.5 lbs. |
| " | $U_{12}$ $L_{12}$ | 12.33 | 2 11 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| End post | $U_{12}$ $L_{13}$ | 39.1 | 1 21 × $\frac{5}{8}$.<br>2 16 × $\frac{7}{16}$.<br>2 L's 3 × 3 × $\frac{7}{16}$.<br>2 L's 4 × 4 × 35.4 lbs. |

260-FOOT CANTILEVER—(*Continued.*)

| Member. | Mark. | Required section. | Material used. |
|---|---|---|---|
| | | Sq. inches. | |
| Web ..... | $L_0$ $M_1$ | 22.0 | 2 15 × $\frac{3}{8}$.<br>4 L's 3 × 3 × 27 lbs. |
| " ..... | $M_1$ $U_2$ | 16.8 | 2 15 × $\frac{3}{8}$.<br>4 L's 3 × 3 × $\frac{5}{16}$. |
| " ..... | $L_2$ $M_3$ | 18.1 | 2 13 × $\frac{3}{8}$.<br>4 L's 3 × 3 × 22.8 lbs. |
| " ..... | $M_3$ $U_4$ | 14.8 | 2 13 × $\frac{5}{16}$.<br>4 L's 3 × 3 × 18 lbs. |
| " ..... | $L_4$ $M_5$ | 15.7 | 2 12 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 20.5 lbs. |
| " ..... | $M_5$ $U_6$ | 12.4 | 2 12 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_6$ $M_7$ | 11.0 | 2 11 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $M_7$ $U_8$ | 10.1 | 2 11 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_8$ $U_9$ | 8.6 | 2 10 × $\frac{5}{16}$.<br>4 $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_9$ $U_{10}$ | 9.7 | 2 10 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_{10}$ $U_{11}$ | 11.0 | 2 10 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_{11}$ $U_{12}$ | 12.8 | 2 11 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| Anchor rods ..... | ..... | .... | 4 bars 2-inch square iron. |

365-FOOT DRAW SPAN.

| | | | |
|---|---|---|---|
| Top chord ..... | $U_0$ $U_0$ | 26.86 | 2 bars 5 × $1\frac{3}{8}$.<br>2 " 5 × $1\frac{5}{16}$. |
| " ..... | $U_0$ $U_2$ | 25.08 | 4 " 5 × $1\frac{1}{4}$. |
| " ..... | $U_2$ $U_4$ | 22.1 | 2 14 × $\frac{7}{16}$.<br>4 L's 3 × 3 × 25.3 lbs. |
| " ..... | $U_4$ $U_5$ | 25.83 | 2 14 × $\frac{1}{2}$.<br>4 L's 3 × 3 × 30 lbs. |
| " ..... | $U_5$ $U_6$ | 26.03 | 2 14 × $\frac{1}{2}$.<br>4 L's 3 × 3 × 30 lbs. |
| " ..... | $U_6$ $U_7$ | 21.98 | 2 14 × $\frac{3}{8}$. |
| " ..... | $U_7$ $U_8$ | 20.07 | 4 L's 3 × 3 × 25 lbs. |
| Bottom chord ..... | $L_0$ $L_0$ | 30.0 | 2 14 × $\frac{1}{2}$ |
| " ..... | $L_0$ $L_2$ | 29.1 | 4 L's $3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{5}{8}$.<br>2 14 × $\frac{3}{8}$. |
| " ..... | $L_2$ $L_4$ | 22.4 | 4 L's 3 × 3 × 29.8 lbs. |
| " ..... | $L_4$ $L_5$ | 23.78 | 2 14 × $\frac{3}{8}$.<br>4 L's 3 × 3 × 32.2 lbs. |
| " ..... | $L_5$ $L_6$ | 21.68 | 2 14 × $\frac{3}{8}$. |
| " ..... | $L_6$ $L_7$ | 21.15 | 4 L's 3 × 3 × 28 lbs. |
| " ..... | $L_7$ $L_8$ | 14.26 | 2 14 × $\frac{3}{8}$. |
| " ..... | $L_8$ $L_9$ | 12.91 | 4 L's 3 × 3 × $\frac{5}{16}$. |
| Web ..... | $U_0$ $L_0$ | 37.4 | 2 18 × $\frac{1}{2}$.<br>4 L's 4 × 4 × 48.5 lbs. iron. |
| " ..... | $U_0$ $M_1$ | 15.67 | 2 bars 5 × $1\frac{1}{16}$ |
| " ..... | $M_1$ $L_2$ | 14.85 | 2 " 4 × 1. |
| " ..... | $U_2$ $M_3$ | 13.70 | 2 " 5 × $1\frac{3}{8}$. |
| " ..... | $M_3$ $L_4$ | 11.67 | 2 " 5 × $1\frac{3}{16}$. |
| " ..... | $U_4$ $L_5$ | 7.88 | 2 " 4 × 1 |
| End post ..... | $U_8$ $L_9$ | 26.2 | 2 14 × $\frac{1}{2}$.<br>4 L's 3 × 3 × 30.5. |
| Suspender ..... | $U_8$ $L_8$ | 6.12 | 2 bars 4 × 1.<br>2 12 × $\frac{3}{8}$. |
| Web ..... | $L_0$ $M_1$ | 16.0 | 4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 17.7. |
| " ..... | $M_1$ $U_2$ | 12.32 | 2 12 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $L_2$ $M_3$ | 13.47 | 2 11 × $\frac{5}{16}$<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × 16.6 lbs. |
| " ..... | $M_3$ $U_4$ | 11.85 | 2 11 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| Suspender ..... | $L_1$ $M_1$ | 6.12 | 2 bars 4 × 1. |
| " ..... | $L_3$ $M_3$ | 6.12 | 2 " 4 × 1. |

240-FOOT SPAN.

| Member. | Mark. | Required section. | Material used. |
|---|---|---|---|
| | | Sq. inches. | |
| Top chord..... | $U_2$ $U_4$ | 32.5 | 1 22 × $\frac{3}{8}$.<br>2 15 × $\frac{3}{8}$.<br>4 L's 4 × 4 × 33 lbs. |
| " ..... | $U_4$ $U_6$ | 40.0 | 1 22 × $\frac{1}{2}$.<br>2 15 × $\frac{9}{16}$.<br>4 L's 4 × 4 × 30.4 lbs. |
| Bottom chord..... | $L_0$ $L_2$ | 22.54 | 4 bars 5 × $1\frac{1}{8}$. |
| " ..... | $L_2$ $L_4$ | 25.9 | 4 " 5 × $1\frac{5}{16}$. |
| " ..... | $L_4$ $L_6$ | 34.0 | 4 " 5 × $1\frac{3}{16}$.<br>2 " 5 × 1. |
| End Post..... | $L_0$ $M_1$ | 37.7 | 1 22 × $\frac{3}{8}$.<br>2 15 × $\frac{9}{16}$.<br>2 L's 4 × 4 × 28.6 lbs.<br>2 L s 4 × 4 × 34.2 lbs. |
| " ..... | $M_1$ $U_2$ | 34.2 | 1 22 × $\frac{3}{8}$.<br>2 15 × $\frac{7}{16}$.<br>4 L's 4 × 4 × 32 lbs. |
| Web ..... | $U_2$ $M_3$ | 11.7 | 2 bars 5 × $1\frac{3}{16}$. |
| " ..... | $M_3$ $L_4$ | 10.0 | 2 " 5 × 1. |
| " ..... | $L_1$ $M_1$ | 6.06 | 2 " $3\frac{1}{2}$ × $\frac{7}{8}$. |
| " ..... | $L_2$ $M_2$ | 6.06 | 2 " $3\frac{1}{2}$ × $\frac{7}{8}$. |
| " ..... | $L_3$ $M_3$ | 6.06 | 2 " $3\frac{1}{2}$ × $\frac{7}{8}$. |

160-FOOT SUSPENDED SPAN.

| | | | |
|---|---|---|---|
| Top chord ..... | $U_8$ $U_9$ | ..... | 2 14 × $\frac{5}{16}$.<br>4 L's $2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$. |
| " ..... | $U_9$ $U_{10}$ | ..... | 2 14 × $\frac{5}{8}$.<br>4 L's 3 × 3 × $\frac{3}{8}$. |
| " ..... | $U_{10}$ $U_{11}$ | ..... | 1 20 × $\frac{5}{16}$.<br>2 14 × $\frac{3}{8}$.<br>4 L's 3 × 3 × $\frac{3}{8}$. |
| " ..... | $U_{11}$ $U_{12}$ | ..... | 1 20 × $\frac{7}{16}$.<br>2 14 × $\frac{3}{8}$.<br>4 L's 3 × 3 × $\frac{3}{8}$. |
| Bottom chord..... | $L_8$ $L_9$ | ..... .. | 2 12 × $\frac{5}{16}$.<br>4 L's 3 × 3 × $\frac{3}{8}$. |
| " ..... ..... | $L_9$ $L_{10}$ | ..... | 2 12 × $\frac{5}{16}$.<br>4 L's 3 × 3 × $\frac{3}{8}$. |
| " ..... | $L_{10}$ $L_{11}$ | ..... | 2 12 × $\frac{7}{16}$.<br>4 L's 3 × 3 × $\frac{9}{16}$. |
| " ..... | $L_{11}$ $L_{12}$ | ..... | 4 bars 5 × $1\frac{1}{8}$.<br>1 18 × $\frac{3}{8}$. |
| Web ..... | $L_8$ $U_9$ | ..... | 2 14 × $\frac{3}{8}$.<br>4 L's 3 × 3 × 19 lbs. |
| | $L_9$ $U_9$ | ..... | 2 bars 4 × $1\frac{1}{8}$. |
| | $L_{10}$ $U_9$ | ..... | 2 " 4 × $1\frac{1}{16}$. |
| | $L_{11}$ $U_{10}$ | ..... | 2 " 3 × 1. |

SUMMARY OF ACTUAL COST OF COMPLETED BRIDGE.

| | | |
|---|---|---|
| Pier No. 1, cylinder pier | | $11 040.27 |
| " 9, " | | 19 221.57 |
| Piers No. 2 to 8, inclusive, masonry piers— | | |
| Estimates paid—stone | $79 349.54 | |
| " freight on stone | 24 602.68 | |
| " laying masonry | 85 253.21 | |
| " and Bridge Company's payrolls, foundations | 53 385.62 | |
| | $242 591.05 | |
| Bridge Company's labor and general expense, net | 22 846.98 | |
| | | 265 438.03 |
| Total substructure | | $295 699.87 |
| " superstructure, net | | 577 200.00 |
| " cost (engineering not included) | | $872 899.87 |

Total number cubic yards masonry = 16 240.

Total cost of masonry per cubic yard in place, $11.67.

Shipping weights of superstructure metal as follows, viz.:

| | Iron, pounds. | Steel, pounds. |
|---|---|---|
| Two cantilevers 260 feet | 508 760 | 898 756 |
| Two cantilever spans 480 and 483 feet | 782 822 | 1 123 222 |
| One supporting span 360 feet | 337 783 | 884 214 |
| Total cantilever system 1 843 feet | 1 629 365 | 2 906 192 |
| One draw span above table | 354 074 | 211 663 |
| Turn-table | 176 625 | 855 |
| One span 238 feet | 256 826 | 183 746 |
| Cylinder pier bracing and miscellaneous | 27 880 | 72 248 |
| Total, not including engine and cast-iron ornaments | 2 444 770 | 3 374 704 |

The engineering organization from beginning to the completion of structure was as follows:

*President*..... BENNETT H. YOUNG.
*Chief Engineer*..... JOHN MACLEOD.

*Resident Engineers.*

PRELIMINARY SURVEYS AND LOCATION.—MACE MOULTON, April 10th, 1881, to June 2d, 1881.

SUBSTRUCTURE.—MACE MOULTON, October 10th, 1881, to May 15th, 1882. Work suspended.

H. P. MCDONALD..... October 1st, 1883, to February 1st, 1884.
C. A. BRADY..... July 1st, 1884, to June 6th, 1885.

*Assistant Engineer.*

J. K. ZOLLINGER..... October 10th, 1881, to June 6th, 1885.

SUPERSTRUCTURE.

*Chief Assistant Engineer.*—MACE MOULTON, February 16th, 1885, to July 15th, 1886.

*Assistant Engineer on Calculations.*—J. W. SCHAUB.

*Inspecting Engineers.*

MATERIAL.—H. GOLDMARK, Cambria Iron Company and Pennsylvania Steel Company.
" G. W. G. FERRIS, Carnegie Brothers & Co.

WORKMANSHIP AND MATERIAL.—W. H. BREITHAUPT, Athens Shop.
" " W. R. WEBSTER, New Jersey Steel and Iron Company's Shop.

Bridge completed July 15th, 1886.

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The following illustrations are given herewith.

Plate XIII. The structure completed.
" XIV. Map showing location of bridge.
" XV. Rise and fall of Ohio River at bridge site.
" XVI. Original design by C. Shaler Smith, and final design by Charles Macdonald and Edward Hemberle.
" XVII. Erection devices for joining cantilevers.
" XVIII. Stress diagrams.
" XIX. Cross section of floor.
" XX. Posts made from tension and compression steel.
" XXI. The structure in progress.

# APPENDIX.

## EXTRACTS FROM SPECIFICATIONS.

### General Description.

The total length of bridge will be 2 453 feet center to center of abutment.

The length of the different spans is given below, commencing at the Kentucky shore.

| | Center to center of piers. |
|---|---|
| Shore span | 260 feet. |
| Span over canal | 483 " |
| " Sand Island | 360 " |
| " main river | 480 " |
| " " | 260 " |
| Draw span, total length | 370 " |
| Indiana shore span | 240 " |
| Total length | 2 453 feet. |

The structure will be built for a single track railroad, two (2) roadways not less than ten (10) feet wide in the clear, and two (2) sidewalks each four (4) feet in width in the clear.

The railroad track to be placed in the center between the two trusses; one roadway on the outside of each truss, supported upon projecting brackets; and the sidewalks one on either side of the railroad track and inside the main trusses.

The clear width between trusses shall not be less than twenty-two (22) feet.

There will be substantial wooden hand-rails on the outer line of the roadways, and between the sidewalks and the railroad track, and close wooden partitions eight (8) feet high between the sidewalks and the roadways.

The entire superstructure will be of wrought-iron and steel, with the exception of the roadway and sidewalk floors, and the ties and guard-rails for the railroad, which will be of timber, as below indicated. Steel will be used for top and bottom chords, end braces, pins, bearing plates and rollers. Longitudinal stringers to be of Southern pine, 7 by 16 inches, spaced about two feet apart.

Lower floor of roadway to be 3-inch oak.

Upper floor of roadway to be 2-inch spruce.

Floor of sidewalks to be 2-inch Southern pine.

Ties to be white oak.

Guard-rails to be yellow pine.

Cast-iron may be used for the construction of the center circular track, wheels, etc., of the turn-table of the draw.

## Proportion of Parts and Details of Construction.

The bridge will be proportioned to carry, in addition to its own weight, the following moving loads, viz.:

For railroad traffic, a train of freight cars weighing 2 240 pounds per foot and drawn by two (2) standard consolidation locomotives weighing 57 tons each; and for the roadway and sidewalk traffic, 1 200 pounds per lineal foot of bridge.

| | Engine. | | | | | Tender. | | | | Train. |
|---|---|---|---|---|---|---|---|---|---|---|
| Load, pounds.... | 16 000 | 24 000 | 24 000 | 24 000 | 24 000 | 16 000 | 16 000 | 16 000 | 16 000 | 2 240 pounds per foot. |
| Feet..... | 7′ 6″ | 4′ 6″ | 4′ 6″ | 4′ 6″ | 10′ 6″ | 5′ 0″ | 5′ 6″ | 5′ 0″ | 8′ 0″ | |

| | Engine. | | | | | Tender. | | | | Train. |
|---|---|---|---|---|---|---|---|---|---|---|
| Load, pounds.... | 16 000 | 24 000 | 24 000 | 24 000 | 24 000 | 16 000 | 16 000 | 16 000 | 16 000 | 2 240 pounds per foot. |
| Feet..... | 7′ 6″ | 4′ 6″ | 4′ 6″ | 4′ 6″ | 10′ 6″ | 5′ 0″ | 5′ 6″ | 5′ 0″ | 3′ 0″ | |

This moving load of engines and train of loaded cars being distributed as show in the foregoing diagram, and considered in positions and conditions giving the greatest results.

To provide for the effect of impact, all such parts of the structure as are liable to be subjected to sudden strains or vibrations shall be calculated with additions to the above specified rolling load as follows:

| | Per cent. |
|---|---|
| Floor-beam hangers and riveted connections of stringers and floor beams | 100 |
| Floor-beam vertical suspenders over 15 feet long | 50 |
| Stringers, floor beams and members subjected to counterstrain | 25 |

The maximum and minimum stresses in compression and tension, as found for the before mentioned loads, with a factor of safety of not less than five, are to be used in determining the permissible working stress in each piece of the structure according to Launhardt's formula, as follows:

$$a = u\left(1 + \frac{\text{minimum stress in member}}{2 \text{ maximum stress in member}}\right)$$

| In the above formula | Pounds per square inch. |
|---|---|
| $a$ = permissible stress per square inch for double refined iron in tension (links or rods) ..... | 9 000 |
| $u$ = for double refined steel in tension..... | 14 000 |
| $u$ = for rolled iron in tension (plates or shapes) . | 8 500 |
| $u$ = for rolled steel in tension..... | 14 000 |
| $u$ = for rolled iron in compression ..... | 7 500 |
| $u$ = for rolled steel in compression ..... | 13 000 |

The permissible stress per square inch for members in compression is to be reduced in proportion to the ratio of the length to the least radius of gyration of the section, by the following formula:

$$\text{For both ends fixed, } b = \frac{a}{1 + \frac{l^2}{40\,000\, r^2}}$$

$$\text{For one end jointed, } b = \frac{a}{1 + \frac{l^2}{30\,000\, r^2}}$$

$$\text{For both ends jointed, } b = \frac{a}{1 + \frac{l^2}{20\,000\, r^2}}$$

When $a$ = permissible stress previously found.
$b$ = the allowable working stress per square inch.
$l$ = length of piece in inches center to center of connections.
$r$ = radius of gyration of the section in inches.

To provide for wind strains and vibration, the bottom lateral bracing shall be proportioned to resist a force at right angles to the axis of the bridge equivalent to 450 pounds per foot of span. For top lateral bracing, 150 pounds per foot of span.

The permissible working strains in the lateral bracing as determined by the above assumption, may be 50 per cent. greater than the values determined by the formula for loads as previously given. But in no case shall any lateral or diagonal rod have a less area than three-fourths of a square inch.

The unsupported width of any plate subjected to compression shall never exceed thirty times its thickness ; nor shall any plate be less than five-sixteenths of an inch thick.

The sections of top chords shall be connected at all abutting joints by splices sufficient to hold them truly in position.

In rolled I beams the compression per square inch in the compression flanges must not exceed $\frac{10\,000}{1 + \frac{l^2}{5\,000\, b^2}}$

In riveted girders,

$$\frac{8\,000}{1 + \frac{l^2}{5\,000\,b^2}}$$

Where $l$ = length of unsupported compressed flange in inches, $b$ = breadth in inches of the compressed flange.

The shearing strain per square inch of I beams and girders must not exceed

$$\frac{8\,000}{1 + \frac{d^2}{3\,000\,t^2}}$$

Where $d$ = distance between flanges or stiffeners measured on a line inclined 45 degrees, and $t$ = thickness of web in inches.

The shearing strain on wrought-iron pins must not exceed 7 000 pounds per square inch, nor on steel pins 10 000 pounds per square inch.

The strain on extreme fibers caused by bending must not exceed 15 000 pounds per square inch for wrought-iron, nor 20 000 pounds per square inch for steel pins, the forces being considered as applied at center of bearing, each surface. The bearing strain on an area equal to the diameter of the pin multiplied by the thickness of the head must not exceed 12 000 pounds per square inch for wrought-iron, nor 18 000 pounds per square inch for steel pins.

The shearing strain on rivets must not exceed 7 000 pounds per square inch if of wrought-iron, nor 10 000 pounds per square inch if of steel. The bearing strain on the surface of rivets (diameter of rivet multiplied by thickness of plate) must not be more than 10 000 pounds per square inch if of wrought-iron, nor more than 15 000 pounds per square inch if of steel, for all rivets used in bearing and splice plates. For rivets in all other positions a maximum bearing strain of 12 000 pounds per square inch if of wrought-iron, and 18 000 pounds per square inch if of steel, will be allowed. Rivets will not be used where they may be subjected to tensile strain. No allowance will be made for countersunk rivets when countersink is in a plate less than ¾ of an inch thick.

The heads of eye-bars must not be inferior in strength to the body of the bar. The form of the head and the mode of manufacture shall be subject to the approval of the engineer.

In compression members the distance from center to center of rivets must not exceed six inches, or sixteen times the thickness of any of the joined plates, and the pitch of rivets for a distance of two diameters from the ends shall be four times the diameters of the rivets; but in no case shall the pitch of the rivets be less than four times the diameters of the rivets.

The sectional area of rivets in one segment, in the distance of two diameters from the end, must not be less than the sectional area of the segment.

Where columns composed of two segments latticed are used, the number of rivets in end tie-plates shall be sufficient to transfer one-half of the strain coming on the column across from one segment to the opposite segment independent of the lattice.

All segments must be of one length, without break, whenever practicable. For trough-shaped columns the number of rivets in lower tie-plate shall be sufficient to transfer one-fourth of the strain coming on the column.

Where lattice-work is used, the angle of the lattice bars with center line of the member shall be about forty-five degrees. The size of the bars shall be proportioned to the width of the members and the strain to which they may be subjected.

The distance between the edge of any piece and center of rivet-hole must never be less than one and one-fourth inches, except in bars less than two and one half inches wide; where practicable it shall be at least two diameters of rivets.

When plates more than twelve inches wide are used in the flanges of riveted girders, an extra line of rivets with a pitch of not less than nine inches shall be driven along each edge to draw the plates together and prevent entrance of water.

All joints in riveted girders, whether in tension or compression, must be fully spliced, as no reliance will be placed on abutting joints. The ends however must be dressed straight and true, so as to leave no open joints. The web of plate girders must be spliced at all joints by a plate on each side of the web.

The diameter of rivets will ordinarily be three-fourths or seven-eighths inch.

Floor-beam suspension bars are to be so arranged by means of equalizers, or otherwise, as to secure equal strains on all links supporting the same floor beam.

Rods with screw ends shall be upset at the ends so as to make the diameter at the bottom of the threads one-eighth inch larger than any part of the body of the bar. The nuts must have a true and square bearing on the surface they rest upon, be easily accessible with a wrench for the purpose of adjustment, and be effectively checked after the final adjustment, as also all pin nuts.

There must be a bearing plate or box of approved form under both ends of all spans of sufficient depth to distribute the weight properly on masonry. These plates must be of such dimensions that the greatest pressure upon the masonry will not exceed 200 pounds per square inch.

All deck and through spans, excepting the channel span, must have at one end nests of turned friction rollers of steel running between planed surfaces. The rollers must not have less than three inches diameter, and shall be so proportioned that the pressure in pounds per lineal inch of rollers shall not exceed $\sqrt{540\ 000\ d}$ ($d$ being the diameter of the rollers in inches).

All spans must be sufficiently anchored to the masonry to resist displacement by the strongest wind specified.

All connections and details of the several parts of the structure shall be of such strength, that upon testing, rupture shall occur in the body of the members rather than in any details or connections.

All surfaces not in contact with other surfaces must be accessible to inspection, cleaning and painting after erection. No closed work will be allowed in the structure.

## Materials.

Steel.—The steel shall be manufactured by the open-hearth process; Bessemer steel will not be accepted. A sample bar three-quarters of an inch in diameter shall be rolled from every melt; if this bar fails to meet the requirements of the laboratory tests the whole charge shall be rejected.

Steel used in compression members, bolsters, bearing plates, pins and rollers shall contain not less than $\frac{34}{100}$ nor more than $\frac{42}{100}$ of one per cent. of carbon, and less than one-tenth per cent. of phosphorus. A sample test bar three-quarters of an inch in. diameter shall bend 180 degrees around its own diameter without sign of crack or flaw. The same bar, tested in a lever machine, shall show an elastic limit of not less than 50 000 pounds and an ultimate strength of not less than 80 000 pounds per square inch; it shall elongate at least 15 per cent. in a length of eight inches before breaking, and shall have a reduced area of 35 per cent. at the point of fracture. It shall be incapable of tempering.

Steel for rivets and eye-bars shall contain not more than $\frac{25}{100}$ of one per cent. of carbon, and less than one-tenth of one per cent. of phosphorus. A sample bar three-quarters of an inch in diameter shall bend 180 degrees and be set back upon itself without showing crack or flaw. When tested in a lever machine it shall have an elastic limit of not less than 40 000 pounds, and an ultimate strength of not less than 70 000 pounds per square inch; it shall elongate at least 18 per cent. in a length of eight inches, and shall show a reduction of at least 42 per cent. at the point of fracture. In full-sized bars this steel shall have an elastic limit of at least 35 000 pounds, and an ultimate strength of at least 65 000 pounds per square inch; it shall elongate at least 10 per cent. before breaking, and for strains less than 30 000 pounds per square inch shall show a modulus of elasticity between 28 000 000 and 30 000 000 pounds.

Facilities for testing the sample bars shall be furnished by the contractor at a point convenient to the steel-works, and tests shall be made at the expense of the contractor, and under the direction of the engineer. All plates for this work of both iron and steel shall be rolled in a universal mill.

Steel for pins shall not be hammered, but rolled between gothic rolls. All rolled beams, channels, bars, angles, plates, etc., must be straight and out of wind. They must be free from flaws, fins, blisters, seams, and cracked edges. Angles and edges must be sharp and well filled out. Flaws, surface imperfections, or irregular shapes will be sufficient ground for the rejection of material

Specifications for iron and timber are as usual in first-class work.

All specifications for workmanship as usual in first-class work; the holes in riveted steel to be punched $\frac{1}{16}$ inch smaller diameter than nominal diameter of rivet and then reamed, with edges of reamed holes filed to take off burr left by drill.

NOTE.—The following changes and supplementary requirements were considered best by the writer, and the work was done in accordance therewith.

Compression steel to be used as specified, except for pins—lower limit of carbon fixed at 0.28 per cent.

Tension steel to be used for eye-bars; also for riveted members subject to tension or alternate stresses; also for pins. This grade was not used for rivets; carbon limits changed to vary between 0.18 and 0.28 per cent.

Rivet steel. Carbon not specified. Ultimate strength to vary between 58 000 and 65 000 pounds per square inch with severe bending and quenching tests.

All rivet holes in steel were reamed except those in lacing bars. All plates were either universal mill or groove-rolled plates. No planing of end of plates was required, except where proper finish required it.

The only case of hammering steel plate allowed was that mentioned in "special tests."

Eye-bars were made by upsetting and head finished under the hammer.

Eye-bar heads proportioned in general with 50 per cent. excess material across eye when pin diameter equaled width of bar, the percentage of excess decreasing as the diameter of pin exceeded the width of bar. Steel lateral bar upsets were made with care, providing against sudden changes in shape and sharp corners. Threads were cut with V threads, with fillet at bottom.

All reaming and drilling was done with rigidly-fixed vertical drills, and parts riveted up without taking apart.

All ends of members where sheared or punched off for clearance requirements were carefully chipped so as to leave no sharp re-entrant angles.

TESTS ON MATERIAL AND FINISHED PARTS.

The various records of tests made on material and finished parts have been placed by themselves in order to separate the descriptive from the special matter and immediately follow in two classes.

1st. *Regular tests* made to determine quality, with view to acceptance or rejection of material.

2d. *Special tests* made for the purpose of determining questions arising during the progress of the work, or with a view to testing the soundness of prevalent theories as applied to the material in hand.

## REGULAR TESTS.

As will be seen from specification the quality of the steel was to be determined by the tests on $\frac{3}{4}''$ rounds rolled from ingots. No further provision was made therein for future rejection on score of quality, and with this we were obliged to be satisfied with the finished eyebar and angle bars made at Johnstown. However, by courtesy of the Union Bridge Company, extra lengths of angle bars and extra bars of eyebar steel were ordered and test pieces cut therefrom which enabled us to know how the steel was running. All tests given are on steel accepted, except where otherwise noted.

From the start the mills were notified that all finished material must have the original heat number stamped or stenciled on in order that future reference might be made to them. In material rolled at Carnegie's, however, the tests were made on crop ends of finished plates to determine state of the material as regards overheating in furnaces in which slabs were heated for rolling into finished plate. Very few rejections occurred, but the series of tests gave us satisfactory assurance of the excellence of the finished product.

I have collected all tests made on material from each original heat, both $\frac{3}{4}''$ round from ingots and finished steel, so that it may be judged how much or how little relation there may be existing between the condition of the two. The tests recorded for plate steel, were made at the Union Iron Mills, Pittsburgh, and the ingot tests at the works of the Penna. Steel Co., at Steelton, and were all made with care.

The work done on the test pieces at Steelton was as follows:—14″ square ingot heated and rolled to 7″ square; 7″ square heated and hammered to 4″ square; 4″ square heated and rolled to $1\frac{3}{4}'' \times 2''$ square; this heated again and rolled to $\frac{3}{4}''$ round test bar. This bar then tested without machine finish. In all cases where modulus of elasticity is recorded, the elongations by which it was determined were taken by an instrument with an electric contact micrometer reading attachment.

Work done on plate material. Plates $\frac{1}{2}''$ thick and under rolled from slabs made 4″thick x $\frac{1}{2}''$ wider than finished plate. These slabs either hammered or rolled in universal mill from ingots. Plates over $\frac{1}{2}''$ thick same, except that slabs were 5″ thick.

Plates over 18″ wide made from 20″x20″ ingots, and for plates over 20″ wide ingots first spread under hammer, and then rolled to required size. Plate test pieces planed from plates—generally 1″ wide.

## PLATES USED IN *TENSION*, OR ALTERNATE *TENSION* AND *COMPRESSION*, ½ INCH THICK AND UNDER.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/11 | 4528 | 15×⅜ | .4821 | 47.080 | 72.910 | 24.0 | 53.0 | 180° round ½″ | | | |
| " | " | 12×7/16 | .4250 | 43.950 | 73.530 | 25.0 | 55.0 | " | | | |
| 5/26 | " | 12×⅜ | .3342 | 50.000 | 74.600 | 22.0 | 57.7 | | | | |
| " | " | 12×7/16 | .3913 | 43.730 | 73.210 | 24.0 | 48.3 | | | | |
| 7/29 | " | 12×⅜ | .3805 | 51.328 | 75.530 | 23.8 | 55.3 | | | | |
| | | | | | | | | From | | | |
| 4/20 | " | ¾ round | ..... | 50.066 | 74.266 | 24.3 | 52.9 | 14″×14″ Ingot | .21 | .71 | |
| " | " | " | ..... | 49.040 | 74.020 | 25.0 | 51.9 | " | " | " | |
| " | " | " | ..... | 46.130 | 71.290 | 24.1 | 57.4 | " | " | " | Test piece annealed. |
| | | | | | | | | No. 1 | | | |
| 5/ | " | " | ..... | 55.660 | 78.700 | 25.0 | 53.3 | 20″×20″ Ingot | .25 | | |
| " | " | " | ..... | 54.270 | 78.500 | 25.0 | 55.3 | " No. 1 | " | | |
| " | " | " | ..... | 51.070 | 74.070 | 25.0 | 58.2 | " No. 2 | .22 | | |
| " | " | " | ..... | 51.560 | 74.700 | 25.7 | 55.5 | " " | " | | |
| " | " | " | ..... | 53.490 | 76.070 | 25.5 | 58.6 | " No. 3 | .21 | | |
| " | " | " | ..... | 52.570 | 74.770 | 26.0 | 59.0 | " " | " | | |
| " | " | " | ..... | 47.320 | 72.180 | 25.5 | 55.5 | " No. 4 | .22 | | |
| " | " | " | ..... | 46.500 | 72.150 | 27.2 | 58.6 | " " | " | | |
| " | " | " | ..... | 46.370 | 73.530 | 24.0 | 52.7 | " No. 5 | .21 | | |
| " | " | " | ..... | 46.120 | 73.750 | 27.0 | 58.8 | " " | " | | |
| " | " | " | ..... | 47.420 | 72.130 | 27.0 | 55.5 | " No. 6 | .21 | | |
| " | " | " | ..... | 47.960 | 70.360 | 26.5 | 56.5 | " " | " | | |
| 5/8 | 4536 | 12×⅜ | .3249 | 46.170 | 78.180 | 21.3 | 52.7 | 180° round ½″ | | | |
| " | " | " | .3478 | 45.740 | 80.790 | 20.6 | 49.7 | " " ⅝″ | | | |
| 6/11 | " | 12×7/16 | .3654 | 46.090 | 77.180 | 21.3 | 50.8 | " " ½″ | | | |
| 6/12 | " | 18×⅜ | .3602 | 48.530 | 75.850 | 18.0 | 38.7 | " " ½″ | ... | .... | Slag pits on surface. |
| 7/17 | " | 14×5/16 | .3187 | 43.645 | 75.458 | 23.0 | 55.6 | 178° " ⅝″ | | | |
| 4/20 | " | ¾ round | ..... | 50.570 | 75.710 | 25.3 | 52.7 | 14″×14″ Ingot | .23 | .89 | |
| " | " | " | ..... | 51.310 | 76.300 | 25.0 | 50.1 | " | " | " | |
| " | " | " | ..... | 47.530 | 72.420 | 21.6 | 56.6 | " | " | " | Test piece annealed. |
| | | | | | | | | No. 1 | | | |
| 5/ | 4536 | ¾ round | ..... | 51.860 | 78.170 | 25.1 | 56.0 | 20″×20″ Ingot | | | |
| " | " | " | ..... | 55.670 | 78.810 | 24.6 | 57.6 | " No. 1 | | | |
| " | " | " | ..... | 50.750 | 76.730 | 25.2 | 54.1 | " No. 2 | | | |
| " | " | " | ..... | 50.870 | 76.860 | 23.9 | 54.0 | " " | | | |
| " | " | " | ..... | 54.100 | 76.220 | 26.2 | 53.4 | " No. 3 | | | |
| " | " | " | ..... | 49.700 | 77.230 | 25.0 | 54.0 | " " | | | |
| " | " | " | ..... | 48.450 | 77.280 | 24.2 | 56.7 | " No. 4 | | | |
| " | " | " | ..... | 48.780 | 78.090 | 25.7 | 57.3 | " " | | | |
| " | " | " | ..... | 47.740 | 74.180 | 27.0 | 54.5 | " No. 5 | | | |
| " | " | " | ..... | 47.520 | 74.490 | 26.2 | 54.6 | " " | | | |
| " | " | " | ..... | 50.600 | 78.820 | 23.9 | 49.4 | " No. 6 | | | |
| " | " | " | ..... | 51.670 | 79.050 | 25.0 | 46.3 | " " | | | |
| 6/11 | 4586 | 18×7/16 | .4300 | 46.050 | 74.120 | 22.5 | 53.1 | 180° round ⅜″ | | | |
| 6/12 | " | 18×⅜ | .3881 | 49.860 | 77.940 | 18.8 | 45.5 | " " ½″ | | | |
| " | " | " | .3596 | 49.600 | 75.530 | 23.0 | 47.3 | | | | |
| 6/25 | " | 18×½ | .5135 | 40.700 | 73.840 | 21.5 | 46.9 | 180° " 7/16″ | | | |
| 7/17 | " | 12×5/16 | .3013 | 46.301 | 71.360 | 23.8 | 54.6 | | | | |
| 5/5 | " | ¾ round | ..... | 49.300 | 75.420 | 24.2 | 49.5 | 14″×14″ Ingot | .19 | 1.01 | |
| " | " | " | ..... | 49.530 | 76.330 | 24.1 | 48.1 | " " | | | |
| 6/11 | 4587 | 18×½ | .5057 | 44.570 | 74.708 | 21.0 | 47.1 | 180° round ⅜″ | | | |
| " | " | " | .5133 | 50.000 | 76.000 | 21.3 | 38.6 | " " ⅝″ | .... | .... | Plate pu'ch'd very hard |
| 5/14 | " | ¾ round | ..... | 46.400 | 72.320 | 25.2 | 51.0 | 14″×14″ Ingot | .21 | .86 | |
| " | " | " | ..... | 50.600 | 76.110 | 25.0 | 48.5 | " " | | | |
| 6/11 | 4590 | 18×⅜ | .3147 | 47.200 | 75.850 | 23.0 | 51.2 | 180° round ½″ | | | |
| 6/25 | " | 18×½ | .5100 | 44.000 | 74.800 | 22.0 | 48.3 | 182° " ⅝″ | .. | .. . | Plate pu'ch'd very hard |
| 0/13 | " | 28×5/16 | .2590 | 49.540 | 77.030 | 14.5 | 55.3 | | | | Broke near jaws |
| 5/14 | " | ¾ round | ..... | 45.790 | 69.150 | 26.7 | 55.5 | 14″×14″ Ingot | .23 | .65 | Fracture irregular. |
| " | " | " | ..... | 46.070 | 69.030 | 27.0 | 55.6 | " " | .... | .... | " " |
| 6/25 | 4593 | 12×5/16 | .3175 | 45.170 | 70.000 | 24.0 | 59.2 | 180° round ⅜″ | | | |
| 5/14 | " | ¾ round | ..... | 48.100 | 71.930 | 24.1 | 43.5 | 14″×14″ Ingot | .18 | .70 | |
| " | " | " | ..... | 47.760 | 71.580 | 25.1 | 43.5 | " " | | | |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/25 | 4649 | 7×⅜ | .3477 | 44.400 | 81.100 | 19.5 | 36.5 | 178° round ½″ | .... | .... | Broke in flaw. |
| 7/17 | " | 14×5/16 | .3245 | 47.763 | 75.495 | 22.0 | 55.5 | 180° " ⅝″ | | | |
| 6/11 | " | ¾ round | ..... | 48.030 | 73.450 | 25.0 | 48.8 | 14″×14″ Ingot | .31 | .75 | |
| " | " | " | ..... | 47.840 | 74.000 | 25.5 | 49.9 | " " | | | |
| 7/27 | 4555 | 15×7/16 | .4382 | 43.360 | 71.008 | 18.0 | 44.6 | 175° round ¾″ | | | |
| 5/5 | " | ¾ round | ..... | 50.140 | 72.950 | 24.0 | 46.5 | 14″×14″ Ingot | .21 | .65 | |
| " | " | " | ..... | 50.750 | 73.730 | 27.7 | 47.1 | " " | | | |
| 7/29 | 4557 | 11×⅜ | .3514 | 51.225 | 75.695 | 21.3 | 53.9 | | | | |
| 8/11 | " | 20×⅜ | .3673 | 42.880 | 72.828 | 21.3 | 56.3 | | | | |
| 5/5 | " | ¾ round | ..... | 46.070 | 70.700 | 24.3 | 48.1 | 14″×14″ Ingot | .20 | .66 | |
| " | " | " | ..... | 47.130 | 71.930 | 23.7 | 47.6 | " " | | | |
| 7/17 | 4561 | 12×7/16 | .4078 | 46.221 | 70.978 | 25.0 | 55.6 | 178° round ¾″ | | | |
| 5/5 | " | ¾ round | ..... | 51.670 | 74.570 | 25.9 | 55.6 | 14″×14″ Ingot | .22 | .65 | |
| " | " | " | ..... | 51.330 | 74.330 | 25.0 | 56.2 | " " | | | |
| 7/27 | 4643 | 16×½ | .4965 | 45.316 | 76.330 | 23.8 | 54.3 | 180° round 1⅛″ | | | |
| 6/20 | " | ¾ round | ..... | 47.490 | 73.170 | 26.0 | 52.2 | 14″×14″ Ingot | .31 | .87 | |
| " | " | " | ..... | 46.150 | 73.280 | 26.0 | 53.1 | " " | | | |
| 7/27 | 4646 | 16×½ | .5015 | 43.423 | 73.805 | 22.5 | 51.5 | | | | |
| 6/11 | " | ¾ round | ..... | 48.010 | 72.430 | 27.0 | 49.0 | 14″×14″ Ingot | .25 | .82 | |
| " | " | " | ..... | 47.010 | 72.470 | 26.3 | 49.9 | " " | | | |
| 6/20 | " | " | ..... | 49.220 | 78.160 | 26.2 | 51.2 | " " | | | |
| " | " | " | ..... | 49.220 | 78.160 | 25.0 | 51.2 | " " | | | |
| 7/29 | 4662 | 13×5/16 | .3065 | 45.695 | 75.370 | 21.3 | 46.9 | | | | |
| 6/20 | " | ¾ round | ..... | 46.400 | 72.520 | 26.0 | 53.8 | 14″×14″ Ingot | .20 | .73 | |
| " | " | " | ..... | 46.400 | 72.320 | 25.2 | 54.0 | " " | | | |
| 8/4 | 4656 | 24×½ | .3911 | 45.000 | 75.556 | 16.3 | 31.8 | ..... | .... | .... | Broke near jaws. |
| 8/26 | " | 24×⅜ | .3486 | 54.415 | 78.595 | 15.5 | 33.2 | ..... | .... | .... | Several surface flaws. |
| 6/20 | " | ¾ round | ..... | 51.620 | 73.890 | 24.5 | 41.4 | 14″×14″ Ingot | .19 | .82 | Test piece much cold rolled. |
| " | " | " | ..... | 51.230 | 73.770 | 25.0 | 41.6 | " " | | | |
| 7/27 | 4645 | 15×⅜ | .3800 | 47.185 | 73.685 | 26.0 | 57.3 | | | | |
| 8/26 | " | 24×⅜ | .3325 | 52.540 | 79.400 | 21.2 | 55.3 | | | | |
| 6/11 | " | ¾ round | ..... | 47.720 | 72.170 | 24.9 | 46.0 | 14″×14″ Ingot | .20 | .76 | |
| " | " | " | ..... | 47.750 | 72.680 | 26.7 | 48.8 | " " | | | |
| 7/17 | 4678 | 7¼×⅜ | .3653 | 45.852 | 71.990 | 23.5 | 52.8 | | | | |
| " | " | 13×5/16 | .2864 | 48.010 | 74.542 | 22.5 | 58.6 | 180° round ¾″ | | | |
| " | " | " | 3110 | 48.230 | 73.150 | 23.0 | 58.0 | 181° round ½″ | | | |
| " | " | 12½×⅜ | .3650 | 43.010 | 71.750 | 23.3 | 53.6 | | | | |
| 6/20 | " | ¾ round | ..... | 48.460 | 73.100 | 26.2 | 54.4 | ..... | .20 | .86 | |
| " | " | " | ..... | 47.960 | 73.520 | 25.5 | 54.7 | | | | |
| 7/27 | 4716 | 15×½ | .4925 | 47.327 | 75.594 | 25.0 | 57.4 | | | | |
| 6/3 | " | ¾ round | ..... | 47.350 | 70.930 | 23.8 | 45.6 | 14″×14″ Ingot | .24 | .86 | |
| " | " | " | ..... | 47.220 | 70.540 | 26.5 | 43.9 | | | | |
| 7/27 | 4720 | 16×½ | .4885 | 45.035 | 71.955 | 23.8 | 51.8 | | | | |
| 7/29 | " | 11×5/16 | .3269 | 52.613 | 77.295 | 23.3 | 53.0 | | | | |
| 6/3 | " | ¾ round | ..... | 46.140 | 70.300 | 24.3 | 45.1 | 14″×14″ Ingot | .21 | .66 | |
| " | " | " | ..... | 46.200 | 70.620 | 25.8 | 45.9 | ..... | .22 | .63 | |
| 8/25 | 4728 | 16×½ | .3871 | 49.470 | 73.110 | 23.8 | 53.3 | | | | |
| 9/8 | " | 20×7/16 | .3336 | 52.250 | 83.150 | 22.5 | 53.4 | | | | |
| 7/22 | " | ¾ round | .... | 50.070 | 78.140 | 24.9 | 49.5 | 14″×14″ Ingot | .26 | .75 | Cold rolled. E=30.-190.000. |
| " | " | " | ..... | 48.990 | 77.010 | 23.0 | 44.3 | " " | | | |
| 9/8 | 4743 | 16×½ | .4000 | 43.995 | 69.990 | 24.3 | 51.5 | | | | |
| 7/31 | " | ¾ round | ..... | 43.360 | 69.280 | 26.3 | 52.9 | ..... | .21 | .65 | |
| " | " | " | ..... | 44.050 | 69.740 | 27.3 | 53.4 | | | | |
| 8/25 | 4738 | 16×½ | .3930 | 47.250 | 70.865 | 26.0 | 52.2 | | | | |
| " | " | 20×5/16 | .2434 | 53.575 | 82.780 | 18.8 | 46.1 | | | | |
| 7/30 | " | ¾ round | ..... | 43.490 | 68.810 | 27.3 | 58.0 | ..... | .19 | .74 | |
| " | " | " | ..... | 45.060 | 68.840 | 27.3 | 58.6 | | | | |
| 12/9 | 4866 | 14×⅜ | .3718 | 43.690 | 74.280 | 25.0 | 55.5 | 180°-close | | | |
| 10/22 | " | " | .3157 | 48.305 | 74.600 | 23.3 | 53.0 | | | | |
| 9/28 | " | ¾ round | ..... | 47.360 | 73.250 | 26.3 | 48.1 | ..... | .20 | | |
| " | " | " | ..... | 48.020 | 73.080 | 24.0 | 46.5 | | | | |
| 12/4 | 4976 | 22½×⅜ | .3656 | 51.420 | 82.400 | 19.4 | 56.6 | 180° round ⅝″ | | | |
| " | " | " | .3694 | 50.620 | 80.230 | 22.0 | 54.4 | 178° " 7/16″ | | | |
| " | " | 25×⅜ | .3781 | 52.100 | 80.000 | 20.0 | 51.2 | 182° " 7/16″ | | | |
| 11/11 | " | ¾ round | ..... | 50.770 | 76.650 | 22.5 | 52.0 | 14″×14″ Ingot | | | |
| " | " | " | ..... | 50.070 | 76.560 | 25.0 | 50.4 | " " | | | |
| 11/13 | " | " | ..... | 53.960 | 77.300 | 22.7 | 47.0 | " " | .25 | .77 | |
| 10/14 | 4776 | 20×7/16 | .3684 | 45.410 | 76.000 | 21.0 | 54.3 | | | | |
| " | " | 17¾×⅜ | .3881 | 46.120 | 75.880 | 23.0 | 54.5 | | | | |
| 8/8 | " | ¾ round | ..... | 47.750 | 75.150 | 24.3 | 46.4 | ..... | .22 | .76 | |
| " | " | " | ..... | 48.590 | 74.940 | 25.0 | 41.1 | | | | |
| 10/24 | 4868 | 13¼×5/16 | .2790 | 47.135 | 74.440 | 23.8 | 50.8 | | | | |
| 9/28 | " | ¾ round | ..... | 49.330 | 74.650 | 23.8 | 49.7 | ..... | .21 | | |
| " | " | " | ..... | 49.350 | 74.720 | 25.0 | 51.6 | | | | |
| 10/26 | 4731 | 13×½ | .4553 | 43.380 | 71.205 | 25.6 | 56.8 | | | | |
| 7/22 | " | ¾ round | ..... | 54.200 | 73.910 | 26.4 | 55.2 | ..... | .... | .... | E=31.160.000. |
| " | " | " | ..... | 51.340 | 71.290 | 26.8 | 47.0 | ..... | .21 | .68 | |

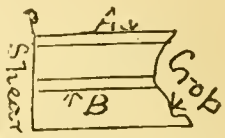
## PLATES USED IN *TENSION* OR ALTERNATE *TENSION* AND *COMPRESSION*—OVER ½ INCH THICK.

| Date, 1885. | Heat No. | Specimen cut from | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/1 | 4519 | 20×⅝ | .4788 | 43.650 | 73.520 | 21.3 | 46.9 | | | | |
| 7/3 | " | 20×¾ | .5625 | 42.280 | 71.336 | 23.8 | 49.3 | 180° round 1⅛″ | | | |
| 4/20 | " | ¾ round | ..... | 49.130 | 73.950 | 25.3 | 49.1 | 14″×14″ Ingot | .22 | .58 | |
| " | " | " | ..... | 49.660 | 74.000 | 23.9 | 49.1 | " | | | |
| 4/21 | " | " | ..... | 46.490 | 70.870 | 23.3 | 53.7 | " | | | |
| 7/3 | 4570 | 20×¾ | .5625 | 42.507 | 69.333 | 22.0 | 43.9 | 180° round 1¾″ | | | |
| " | " | " | .4785 | 41.190 | 68.340 | 20.8 | 40.8 | 180° round 1½″ | | | |
| 5/5 | " | ¾ round | ..... | 46.280 | 70.530 | 25.2 | 49.1 | 14″×14″ Ingot | .23 | .71 | |
| " | " | " | ..... | 46.880 | 71.130 | 24.7 | 49.0 | " | | | |
| 7/1 | 4557 | 20×¾ | .4125 | 43.636 | 78.930 | 18.3 | 42.3 | | | | |
| 7/3 | " | " | .5476 | 41.510 | 68.660 | 20.8 | 40.8 | 180° round 1½″ | .... | .... | Broke in scale pits. |
| 5/5 | " | ¾ round | ..... | 46.070 | 70.700 | 24.3 | 48.1 | 14″×14″ Ingot | .20 | .66 | |
| " | " | " | ..... | 47.130 | 71.930 | 23.7 | 47.6 | " | | | |
| 7/1 | 4561 | 20×¾ | .4706 | 44.470 | 71.630 | 21.3 | 48.8 | 180° round 1¼″ | .... | .... | Showed blow holes. |
| " | " | " | .4703 | 43.590 | 71.870 | 22.5 | 41.8 | | | | |
| 5/5 | " | ¾ round | ..... | 51.670 | 74.570 | 25.9 | 55.6 | 14″×14″ Ingot | .22 | .65 | |
| " | " | " | ..... | 51.330 | 74 330 | 25.0 | 56.2 | " | | | |
| 8/3 | 4588 | 20×⅝ | .5293 | 44.455 | 72.170 | 23.8 | 46.5 | | | | |
| 5/14 | " | ¾ round | ..... | 50.250 | 76.350 | 25.3 | 43.6 | 14″×14″ Ingot | .27 | .78 | |
| " | " | " | ..... | 50.420 | 76.510 | 24.0 | 41.8 | " | | | |
| 7/1 | 4590 | 20×¾ | .4556 | 43.920 | 76.240 | 25.0 | 47.9 | | | | |
| 5/14 | " | ¾ round | ..... | 45.790 | 69.150 | 26.7 | 55.5 | 14″×14″ Ingot | .23 | .65 | |
| " | " | " | ..... | 46.070 | 69.030 | 27.0 | 55.6 | " | | | |
| 6/11 | 4587 | 18×⅝ | .5990 | 41.240 | 70.950 | 24.5 | 45.7 | 180° round ¾″ | | | |
| " | " | 18×9/16 | .4950 | 43.780 | 72.464 | 23.4 | 48.9 | 180° round ⅝″ | ... | .... | Punched hard in spots. |
| " | " | " | .4873 | 41.040 | 75.100 | 22.0 | 46.8 | " | .... | .... | " |
| 5/14 | " | ¾ round | ..... | 46.400 | 72.320 | 25.2 | 51.0 | 14″×14″ Ingot | .21 | .86 | |
| " | " | " | ..... | 50.600 | 76.110 | 25.0 | 48.5 | " | | | |
| 6/11 | 4586 | 18×⅝ | .6047 | 43.000 | 73.750 | 22.0 | 44.0 | 180° round 9/16″ | | | |
| 5/5 | " | ¾ round | ..... | 49.300 | 75.420 | 24.2 | 49.5 | 14″×14″ Ingot | .19 | 1.01 | |
| " | " | " | ..... | 49.530 | 76.330 | 24.1 | 48.4 | " | | | |
| 8/4 | 4721 | 20×⅝ | .4406 | 43.258 | 66.275 | 23.8 | 48.8 | ..... | .... | ... | Broke in surface flaw. |
| " | " | " | .5562 | 42.251 | 66.253 | 23.3 | 50.9 | ..... | .... | .... | " bad " |
| 7/29 | " | " | .6076 | 47.235 | 70.470 | 26.3 | 47.9 | | | | |
| " | " | " | .6139 | 46.733 | 73.500 | 20.0 | 45.8 | | | | |
| 7/3 | " | ¾ round | ..... | 46.920 | 70.260 | 25.7 | 43.3 | 14″×14″ Ingot | .20 | .72 | |
| " | " | " | ..... | 45.780 | 70.370 | 23.0 | 42.3 | " | | | |
| 8/17 | 4720 | 20×⅝ | .6075 | 41.810 | 74.760 | 21.0 | 49.1 | ..... | .... | ... | When punched showed |
| 8/4 | " | " | .1431 | 46.942 | 73.460 | 22.0 | 52.3 | ..... | .... | ... | soft spots. |
| 6/3 | " | ¾ round | ..... | 46.140 | 70.300 | 24.3 | 45.1 | 14″×14″ Ingot | .22 | .63 | |
| " | " | " | ..... | 46.200 | 70.620 | 25.8 | 45.9 | " | | | |
| 7/27 | 4682 | 15×9/16 | .5609 | 44.213 | 72.028 | 20.3 | 49.9 | | | | |
| 8/3 | " | 20×¾ | .6497 | 40.957 | 74.465 | 22.5 | 41.6 | | | | |
| 6/29 | " | ¾ round | ..... | 48.540 | 73.500 | 25.1 | 52.9 | 14″×14″ Ingot | .20 | .81 | |
| " | " | " | ..... | 48.910 | 74.230 | 26 2 | 52.7 | " | | | |
| 7/29 | 4718 | 13×9/16 | .5183 | 43.459 | 68.482 | 25.8 | 52.4 | | | | |
| " | " | " | .5351 | 43.600 | 70.730 | 23.3 | 50.6 | | | | |
| " | " | 16×⅝ | .5177 | 44.651 | 71.130 | 29.0 | 54.4 | | | | |
| 8/26 | " | 20×1⅝ | .5026 | 37.800 | 69.250 | 24.5 | 51.9 | ..... | .... | .... | Specimen round. |
| 6/3 | " | ¾ round | ..... | 46.960 | 70.260 | 23.6 | 42.4 | 14″×14″ Ingot | .23 | .65 | |
| " | " | " | ..... | 47.210 | 70.430 | 24.3 | 41.3 | " | | | |
| 8/25 | 4732 | 24×¾ | .4858 | 41.555 | 71.180 | 17.5 | 41.9 | ..... | .... | .... | Bad surface flaw. |
| 12/9 | " | 9×¾ | .6309 | 43.690 | 71.280 | 23.3 | 46.0 | 180° round ¼″ | | | |
| " | " | 7×1⅛ | .5281 | 40.950 | 71.620 | 27.0 | 53.4 | 180° round ⅛″ | .... | .... | Specimen round. |
| 7/22 | " | ¾ round | ..... | 47.580 | 70.140 | 22.1 | 49.8 | 14″×14″ Ingot | .21 | .72 | E=29.700.000. |
| " | " | " | ..... | 46.920 | 69.030 | 23.3 | 47.2 | " | | | |
| 8/11 | 4733 | 20×¾ | .5437 | 49.767 | 71.180 | 25.0 | 48.5 | | | | |
| 7/22 | " | ¾ round | ..... | 54.570 | 74.380 | 27.2 | 46.1 | 14″×14″ Ingot. | .19 | .58 | Cold rolled specimen. E=29.700.000 |
| " | " | " | ..... | 51.600 | 72.050 | 26.0 | 41.2 | " " | | | |
| 8/11 | 4646 | 20×1⅝ | .4989 | 39.490 | 74.263 | 23.3 | 46.1 | ..... | .... | .... | Specimen turned round |
| 8/26 | " | 20×1¼ | .5026 | 43.590 | 72.120 | 21.5 | 50.0 | ..... | ... | .... | " " |
| 6/11 | " | ¾ round | ..... | 48.010 | 72.430 | 27.0 | 49.0 | 14″×14″ Ingot | .25 | .82 | |

PLATES USED IN *TENSION* OR ALTERNATE *TENSION* AND *COMPRESSION.— Continued.*

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, % | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/11 | 4646 | ¾ round | ..... | 47.010 | 72.470 | 26.3 | 49.9 | | | | |
| 6/20 | " | " | ..... | 49.220 | 78.160 | 26.2 | 51.2 | | | | |
| " | " | " | ..... | 49.220 | 78.160 | 25.0 | 51.2 | | | | |
| 8/3 | 4656 | 24×¾ | .5464 | 41.910 | 72.969 | 23.8 | 42.3 | | | | |
| " | " | " | .6318 | 42.292 | 71.747 | 23.8 | 42.4 | | | | |
| 6/20 | " | ¾ round | ..... | 51.620 | 73.890 | 24.5 | 41.4 | 14″×14″ Ingot | .19 | .82 | Cold rolled specimen. |
| " | " | " | ..... | 51.230 | 73.770 | 25.0 | 41.6 | " | | | |
| 7/29 | 4645 | 16×13/16 | .5635 | 42.462 | 71.418 | 26.0 | 45.9 | | | | |
| 8/3 | " | 21×⅝ | .5345 | 45.444 | 71.693 | 21.9 | 46.8 | | | | |
| 8/4 | " | 24½×⅝ | .5354 | 42.305 | 75.327 | 21.5 | 53.1 | | | | |
| 6/11 | " | ¾ round | ..... | 47.720 | 72.170 | 24.9 | 46.0 | 14″×14″ Ingot | .20 | .76 | |
| " | " | " | ..... | 47.750 | 72.680 | 26.7 | 48.8 | " | | | |
| 8/25 | 4731 | 24×11/16 | .4256 | 46.452 | 70.065 | 21.0 | 56.2 | | | | |
| 9/8 | " | 15×¾ | .6046 | 43.010 | 69.832 | 27.5 | 56.0 | | | | |
| 7/22 | " | ¾ round | ..... | 54.200 | 73.910 | 26.4 | 55.2 | 14″×14″ Ingot | .20 | .68 | E=31.160.000 |
| " | " | " | ..... | 51.340 | 71.290 | 26.8 | 47.0 | " | | | |
| 8/25 | 4738 | 16×9/16 | .3733 | 47.300 | 70.580 | 25.0 | 52.0 | | | | |
| " | " | " | .3699 | 42.824 | 68.400 | 26.0 | 52.2 | | | | |
| 9/8 | " | " | .4510 | 44.094 | 69.660 | 24.5 | 53.1 | | | | |
| " | " | " | .4521 | 43.970 | 69.710 | 25.0 | 56.1 | | | | |
| " | " | 16×⅝ | .5147 | 38.800 | 66.923 | 21.3 | 57.8 | | | | |
| 7/30 | " | ¾ round | ..... | 43.490 | 68.810 | 27.3 | 58.0 | ..... | .19 | .74 | |
| " | " | " | ..... | 45.060 | 68.840 | 27.3 | 58.6 | | | | |
| 8/25 | 4728 | 13½×⅝ | .4174 | 44.682 | 71.750 | 23 8 | 47 9 | | | | |
| 7/22 | " | ¾ round | ..... | 50.070 | 78.140 | 24.9 | 49.5 | 14″×14″ Ingot | .26 | .75 | Cold rolled specimen. E=30.190.000 |
| " | " | " | ..... | 48.990 | 77.010 | 23.0 | 44.3 | " | | | |
| 9/8 | 4772 | 11½×11/16 | .5095 | 38.760 | 69.275 | 25.0 | 55.8 | | | | |
| " | " | 8½×⅝ | .6150 | 41.835 | 76.995 | 23.5 | 50.9 | | | | |
| " | " | 8½×⅞ | .4693 | 43.895 | 75.860 | 22.5 | 53.5 | ..... | .... | .... | Specimen turned round |
| 8/6 | " | ¾ round | ..... | 48.980 | 72.420 | 22.3 | 38.5 | ..... | .20 | .85 | |
| " | " | " | ..... | 48.830 | 72.550 | 24.0 | 38.3 | | | | |
| " | " | " | ..... | 48.630 | 72.720 | 23.0 | 40.2 | | | | |
| " | " | " | ..... | 48.500 | 73.290 | 24.5 | 39.8 | | | | |
| 9/8 | 4774 | 17×¾ | .5964 | 41.752 | 70.004 | 26.3 | 53.0 | | | | |
| 8/6 | " | ¾ round | ..... | 48.480 | 75.620 | 24.5 | 42.2 | ..... | .23 | .78 | |
| " | " | " | ..... | 48.290 | 75.590 | 22.5 | 41.1 | | | | |
| 12/4 | 4868 | 11×⅝ | .6100 | 45.490 | 73.755 | 26.5 | 58.9 | | | | |
| 9/28 | " | ¾ round | ..... | 49.330 | 74.650 | 23.8 | 49.7 | ..... | .21 | | |
| " | " | " | ..... | 49.350 | 74.720 | 25.0 | 51.6 | | | | |
| 10/13 | 4780 | 16×⅝ | .5608 | 42.740 | 75.250 | 23.8 | 46.3 | | | | |
| 8/10 | " | ¾ round | ..... | 49.920 | 76.520 | 22.5 | 39.2 | ..... | .24 | .84 | |
| " | " | " | ..... | 50.820 | 76.560 | 21.9 | 41.3 | | | | |
| 8/26 | 4662 | 20×13/16 | .4717 | 44.240 | 73.140 | 25.0 | 56.7 | ..... | .... | .... | Specimen turned round |
| 6/20 | " | ¾ round | ..... | 46.400 | 72.520 | 26.0 | 53.8 | 14″×14″ Ingot | .20 | .73 | |
| " | " | " | ..... | 46.400 | 72.320 | 25.2 | 54.0 | " | | | |

## PLATES USED IN *COMPRESSION*—½ INCH THICK AND UNDER.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/12 | 4336 | 15×⅜ | ..... | 52.200 | 82.470 | 17.5 | 30.3 | 180° round ⅝″ | .... | .... | Blow hole in center surrounded by granular specks. |
| 4/20 | " | " | ..... | 51.044 | 88.180 | 16.5 | 43.9 | 180° " ⅞″ | | | |
| 4/28 | " A | 13¾×⅜ | ..... | 52.547 | 78.890 | 22.5 | 45.6 | 175° " ⅞″ | .... | .... |  |
| " | " B | " | ..... | 51.800 | 85.570 | 18.0 | 23.9 | 180° " ¾″ | | | |
| 2/ | " | ¾ round | ..... | 48.300 | 86.170 | 20.6 | 34.0 | ..... | .38 | .71 | Test by Fairbanks, Phila. (F) |
| 2/ | " | " | ..... | ..... | 85.980 | 22.8 | 32.7 | ..... | .... | .... | Test at Union Iron Mills, Pittsburgh (U) |
| 2/12 | " | " | ..... | 52.850 | 86.580 | 20.7 | 37.6 | ..... | .... | .... | Test at Penna. S. Co., Steelton. (S) |
| 4/27 | 4341 | 14×⅜ | .3148 | 57.115 | 92.470 | 18.3 | 46.5 | 178° round ⅞″ | | | |
| 2/ | " | ¾ round | ..... | 48.206 | 84.035 | 23.2 | 40.4 | ..... | .31 | .79 | (F) |
| 2/ | " | " | ..... | ..... | 85.650 | 23.3 | 37.9 | ..... | .... | .... | (U) |
| 2/12 | " | " | ..... | 52.000 | 86.740 | 21.9 | 40.3 | ..... | .... | .... | (S) |
| 6/12 | 4342 | 15×⅜ | .3770 | 52.060 | 81.430 | 20.5 | 46.9 | 180° round ½″ | | | |
| 2/ | " | ¾ round | ..... | 51.915 | 82.264 | 15.6 | 22.9 | ..... | .34 | .81 | Broke in punch marks. (F) |
| 2/ | " | " | ..... | ..... | 81.200 | 23.2 | 34.2 | ..... | .... | .... | (U) |
| 2/12 | " | " | ..... | 53.720 | 82.720 | 22.3 | 45.6 | ..... | .... | .... | (S) |
| 4/27 | 4348 | 12×5/16 | .2648 | 47.810 | 85.275 | 18.0 | 48.8 | | | | |
| 5/8 | " | 11×⅜ | .3307 | 56.120 | 83.156 | 20.3 | 45.7 | | | | |
| 2/ | " | ¾ round | ..... | 49.210 | 85.328 | 22.0 | 42.6 | ..... | .31 | 1.02 | (F) |
| 2/ | " | " | ..... | 55.180 | 84.960 | 21.5 | 38.2 | ..... | .... | .... | (S) |
| 8/11 | 4527 | 15×⅜ | .3825 | 49.726 | 86.280 | 18.0 | 47.2 | | | | |
| 4/20 | " | ¾ round | ..... | 52.860 | 83.640 | 21.3 | 40.3 | 14″×14″ Ingot | .29 | .79 | |
| 4/20 | " | " | ..... | 53.480 | 83.420 | 21.8 | 38.1 | " " | | | |
| 4/21 | " | " | ..... | 50.810 | 79.260 | 19.0 | 49.9 | " " | .... | .... | Specimen annealed. |
| 8/11 | 4540 | 13×5/16 | .3068 | 49.480 | 77.120 | 22.0 | 51.3 | | | | |
| " | " | 16×½ | .4885 | 50.890 | 85.142 | 19.3 | 50.9 | | | | |
| 4/20 | " | ¾ round | ..... | 54.060 | 85.980 | 23.0 | 46.4 | 14″×14″ Ingot | .35 | .77 | |
| " | " | " | ..... | 53.080 | 85.080 | 23.0 | 44.5 | " " | | | |
| 6/12 | 4532 | 15×⅜ | .4750 | 53.537 | 84.950 | 21.5 | 37.2 | 180° round ⅝″ | | | |
| " | " | 8¾×7/16 | ..... | 51.670 | 85.580 | 20.0 | 51.6 | | | | |
| 7/17 | " | 11×7/16 | .4108 | 46.005 | 83.730 | 21.0 | 51.0 | | | | |
| 6/26 | " | 9×9/16 | .5232 | 50.360 | 83.330 | 21.3 | 44.4 | 180° round ¾″ | | | |
| 5/ | " | | | | | | | No. 1 | | | |
| | | ¾ round | ..... | 54.000 | 85.780 | 21.3 | 41.8 | 20″×20″ Ingot | | | |
| " | " | " | ..... | 53.070 | 86.300 | 23.2 | 44.8 | " " | | | |
| " | " | " | ..... | 53.930 | 83.760 | 24.6 | 56.4 | " No. 2 | | | |
| " | " | " | ..... | 54.720 | 83.420 | 24.0 | 54.7 | " " | | | |
| 4/20 | " | " | ..... | 53.950 | 82 500 | 23.0 | 52.0 | 14″×14″ Ingot | .33 | .79 | |
| " | " | " | ..... | 54.770 | 82.740 | 24.3 | 49.5 | " " | | | |
| 4/21 | " | " | ..... | 49.230 | 77.580 | 19.8 | 58.3 | " " | .... | .... | Specimen annealed. |
| 5/8 | 4533 | 15×½ | .4343 | 48.880 | 83.810 | 20.0 | 50.3 | 180° round ½″ | | | |
| " | " | " | .4395 | 48.123 | 85.440 | 21.5 | 47.9 | " " ⅝″ | | | |
| 6/12 | " | " | .4882 | 46.300 | 77.370 | 17.0 | 32.3 | " " " | .... | .... | Broke in flaw. |
| 5/8 | " | 15×⅜ | .3511 | 55 916 | 87.550 | 20.8 | 46.1 | " " ⅞″ | | | |
| 4/20 | " | ¾ round | ..... | 52.530 | 82.980 | 23.0 | 45.3 | 14″×14″ Ingot | .35 | .83 | |
| " | " | " | ..... | 53.525 | 81.100 | 22.5 | 44.9 | " " | | | |
| 4/21 | " | " | ..... | 49.690 | 80.170 | 22.0 | 51.2 | " " | .... | .... | Specimen annealed. |
| 6/25 | 4539 | 14×⅜ | .3814 | 50.000 | 83.000 | 20.0 | 36.0 | 175° round ⅝″ | | | |
| 4/28 | " | ¾ round | ..... | 51.880 | 80.830 | 23.3 | 48.1 | 14″×14″ Ingot | .34 | .78 | |
| " | " | " | ..... | 51.910 | 80.660 | 23.9 | 48.1 | " " | | | |
| 6/26 | 4542 | 14×⅜ | .3994 | 47.770 | 81.300 | 18.8 | 53.8 | 180° round ⅞″ | | | |
| " | " | " | .3701 | 50.000 | 80.720 | 20.3 | 51.2 | " ¾ " | | | |
| 4/28 | " | ¾ round | ..... | 55.970 | 86.140 | 22.3 | 46.5 | 14″×14″ Ingot | .37 | .85 | |
| " | " | " | ..... | 54.380 | 86.060 | 23.0 | 45.1 | " " | | | |
| 6/25 | 4546 | 8×⅜ | .3561 | 48.360 | 82.100 | 19.0 | 15.9 | 180° round ¾″ | | | |

## PLATES USED IN *COMPRESSION.—Continued.*

| Date, 1885. | Heat No. | Specimen cut from, | Area of Specimen, | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength; Pounds per Sq. Inch. | Elongation, % in 8″, | Reduction of Area, %. | Bending and Ingot Note., | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4/28 | 4546 | ¾ round | ..... | 51.060 | 81.280 | 23.8 | 47.6 | 14″×14″ Ingot | .34 | .77 | |
| " | " | " | ..... | 50.800 | 80.320 | 24.8 | 46.2 | " " | | | |
| 8/11 | 4547 | 20¼×⅜ | .3645 | 51.083 | 79.010 | 23.3 | 55.8 | | | | |
| 8/25 | " | " | .3664 | 55.500 | 79.635 | 20.3 | 56.3 | | | | |
| 6/12 | " | 20×5/16 | .3048 | 51.900 | 75.555 | 22.5 | 38.3 | 180° round ⅜″ | .. | .... | Broke in surface flaw. |
| 5/26 | " | " | .2822 | 52.000 | 80.000 | 22.0 | 54.4 | | | | |
| 4/28 | " | ¾ round | ..... | 53.770 | 85.620 | 22.6 | 45.7 | 14″×14″ Ingot | .35 | .90 | |
| " | " | " | ..... | 55.400 | 86.040 | 22.3 | 47.2 | " | | | |
| 6/12 | 4548 | 20×⅜ | .3712 | 50.760 | 80.000 | 22.0 | 39.3 | | | | |
| 4/28 | " | ¾ round | ..... | 49.220 | 78.520 | 24.5 | 50.2 | 14″×14″ Ingot | .36 | .78 | |
| " | " | " | ..... | 49.300 | 79.440 | 23.6 | 50.2 | | | | |
| 6/12 | 4550 | 20×5/16 | .3409 | 54.970 | 85.680 | 15.0 | 33.1 | ..... | ... | . | Broke in surface flaw. |
| 4/28 | " | ¾ round | ..... | 50.560 | 79.350 | 24.9 | 47.7 | 14″×14″ Ingot | .34 | .79 | |
| " | " | " | ..... | 50.120 | 79.200 | 23.5 | 47.7 | " | | | |
| 6/26 | 4559 | 14×7/16 | .3973 | 50.170 | 82.380 | 21.2 | 50.0 | 180° round ¾″ | | | |
| " | " | 18×7/16 | .3927 | 49.230 | 80.520 | 22.0 | 40.0 | " round ⅞″ | | | |
| 8/25 | " | 14×⅜ | .2920 | 52.290 | 84.078 | 23.3 | 52.3 | | | | |
| 7/3 | " | 18×7/16 | .4409 | 50.830 | 81.991 | 23.3 | 44.6 | 180° round ¾″ | | | |
| " | " | " | .3925 | 53.660 | 83.580 | 19.5 | 28.6 | " round ⅞″ | | | |
| " | " | 14×½ | .4516 | 50.000 | 76.390 | 23.3 | 46.3 | | | | |
| 5/5 | " | ¾ round | ..... | 52.925 | 80.870 | 22.2 | 45.8 | 14″×14″ Ingot | .36 | .70 | |
| " | " | " | ..... | 52.080 | 81.380 | 22.9 | 49.1 | " | | | |
| 8/25 | 4603 | 19×½ | .3861 | 48.150 | 80.290 | 21.0 | 56.7 | | | | |
| " | " | 22½×½ | .3911 | 47.070 | 80.150 | 24.5 | 55.7 | | | | |
| 5/25 | " | ¾ round | ..... | 56.010 | 81.420 | 23.6 | 48.3 | 14″×14″ Ingot | .25 | .88 | |
| " | " | " | ..... | 55.360 | 81.340 | 24.3 | 51.2 | " | | | |
| 8/17 | 4668 | 16×7/16 | .3161 | 55.330 | 86.088 | 22.5 | 55.4 | | | | |
| 9/8 | " | 15×⅜ | .2983 | 56.755 | 83.473 | 23.5 | 52.9 | | | | |
| 6/11 | " | ¾ round | ..... | 55.430 | 88.040 | 21.9 | 41.1 | 14″×14″ Ingot | .35 | .80 | |
| 6/30 | " | " | ..... | 54.590 | 88.120 | 22.2 | 48.4 | " | | | |
| 7/1 | 4618 | 26×⅜ | .3351 | 58.000 | 89.950 | 16.3 | 43.9 | 165° round ⅞″ | | | |
| " | " | 21×⅜ | .3172 | 58.640 | 91.740 | 17.5 | 44.4 | 180° round ⅝″ | | | |
| " | " | 15×⅜ | .3375 | 58.666 | 89.900 | 18.0 | 47.8 | 180° round ⅞″ | | | |
| 6/25 | " | " | .3853 | 54.800 | 86.400 | 22.0 | 51.5 | 180° round ⅝″ | | | |
| 5/25 | " | ¾ round | ..... | 54.260 | 86.260 | 24.5 | 48.1 | 14″×14″ Ingot | .33 | | |
| " | " | " | ..... | 55.660 | 86.740 | 23.3 | 48.7 | " | | | |
| 8/11 | 4647 | 16×7/16 | .4244 | 44.415 | 85.510 | 20.8 | 51.0 | | | | |
| 6/11 | " | ¾ round | ..... | 48.810 | 82.380 | 20.5 | 34.4 | 14″×14″ Ingot | .38 | .93 | |
| " | " | " | ..... | 49.420 | 82.500 | 21.1 | 41.1 | " | | | |
| 6/5 | " | " | ..... | 51.825 | 84.760 | 23.1 | 39.4 | " | | | |
| " | " | " | ..... | 51.740 | 86.040 | 20.7 | 42.4 | " | | | |
| 12/4 | 4901 | 14×½ | .4990 | 50.260 | 78.160 | 21.3 | 56.6 | 180° round ⅝″ | | | |
| 10/22 | " | 12×⅜ | .3149 | 43.350 | 85.600 | 17.8 | 52.3 | ..... ..... | .... | .... | Broke near jaw. |
| " | " | 14×½ | .4349 | 48.290 | 83.580 | 19.4 | 50.0 | | | | |
| 10/22 | 4600 | 14×½ | .4259 | 43.908 | 82.030 | 23.0 | 51.2 | | | | |
| " | " | " | .4311 | 49.685 | 81.035 | 20.0 | 51.8 | | | | |
| 10/9 | " | ¾ round | ..... | 59.100 | 83.880 | 22.4 | 42.3 | | | | |
| " | " | " | ..... | 56.940 | 82.200 | 24.1 | 44.5 | | | | |
| 12/9 | 4601 | 14×½ | .4990 | 50.415 | 82.664 | 22.0 | 55.8 | 175° round ⅝″ | | | |
| 10/24 | " | 15×⅜ | .3459 | 51.460 | 84.800 | 22.5 | 52.0 | | | | |
| 10/9 | " | ¾ round | ..... | 54.080 | 82.700 | 22.6 | 39.9 | | .32 | | |
| " | " | " | ..... | 54.700 | 82.260 | 21.1 | 37.9 | | | | |
| 10/13 | 4543 | 11×⅜ | .3380 | 55.030 | 84.900 | 20.5 | 47.6 | | | | |
| " | " | " | .3365 | 49.600 | 80.000 | 21.3 | 50.6 | | | | |
| 4/20 | " | ¾ round | ..... | 54.125 | 84.500 | 23.0 | 45 6 | | .32 | .64 | |
| " | " | " | ..... | 53.900 | 84.320 | 22.5 | 45 0 | | | | |
| 10/15 | 4651 | 13×⅜ | .3422 | 50.906 | 81.240 | 20.3 | 54.2 | | | | |
| 10/18 | " | " | .3340 | 51.290 | 81.740 | 18.0 | 47.2 | | | | |
| 6/11 | " | ¾ round | ..... | 48.810 | 82.180 | 24.4 | 53.2 | 14″×14″ Ingot | .34 | .85 | |
| " | " | " | ..... | 52.050 | 82.620 | 23.8 | 50.2 | " | | | |
| 10/26 | " | 20×7/16 | .3784 | 43.410 | 74.260 | 23.0 | 51.9 | | | | |
| " | " | 15×½ | .4337 | 45.420 | 80.820 | 18.8 | 53.5 | | | | |

## PLATES USED IN *COMPRESSION*—OVER ½ INCH THICK.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inches. | Ultimate Strength: Pounds per sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 8/11 | 4527 | 20×⅝ | .4495 | 47.936 | 83.575 | 18.8 | 49.4 | | | | |
| 8/17 | " | " | 4419 | 49.715 | 83.910 | 22.5 | 41.8 | | | | |
| 4/20 | " | ¾ round | ..... | 52.860 | 83.640 | 21.3 | 40.3 | 14″×14″ Ingot | .29 | .79 | |
| " | " | " | ..... | 53.480 | 83.420 | 21.8 | 38.1 | " | | | |
| 4/21 | " | " | ..... | 50.810 | 79.260 | 19.0 | 49.9 | " | .... | .... | Specimen annealed. |
| 5/26 | 4543 | 20×⅝ | .4587 | 47.960 | 75.430 | 24.5 | 54.3 | 180° round ⅜″ | | | |
| 4/28 | " | ¾ round | ..... | 54.125 | 84.500 | 23.0 | 45.6 | 14″×14″ Ingot | .32 | .64 | |
| " | " | " | ..... | 53.900 | 84.320 | 22.5 | 45.0 | " | | | |
| 5/26 | 4538 | 20×¾ | .5031 | 49.000 | 80.000 | 21.3 | 47.1 | 180° round ⅝″ | | | |
| 4/28 | " | ¾ round | ..... | 52.225 | 80.000 | 22.3 | 46.3 | 14″×14″ Ingot | .32 | .95 | |
| " | " | " | ..... | 53.680 | 79.920 | 24.6 | 45.3 | " | | | |
| " | 4539 | 20×⅝ | .4471 | 48.535 | 76.160 | 25.0 | 53.3 | 180° round ½″ | | | |
| 5/26 | " | ¾ round | ..... | 51 880 | 80.830 | 23.3 | 48.1 | 14″×14″ Ingot | .34 | .78 | |
| 4/28 | " | " | ..... | 51.910 | 80.660 | 23.9 | 48.1 | " | | | |
| 8/11 | 4540 | 20×11/16 | .5065 | 46.000 | 79.064 | 16.3 | 22.6 | ..... ..... | .... | .... | Broke in bad surface [flaw. |
| 5/26 | " | 20×¾ | .5058 | 48.616 | 79.700 | 22.0 | 45.8 | 180° round ½″ | | | |
| 4/20 | " | ¾ round | ..... | 54.060 | 85.980 | 23.0 | 46.4 | 14″×14″ Ingot | .35 | .77 | |
| " | " | " | ..... | 53.080 | 85.080 | 23.0 | 44.5 | " | | | |
| 5/26 | 4542 | 20×11/16 | .5425 | 50.690 | 84.516 | 19.5 | 42.3 | 175° round ¾″ | | | |
| 4/28 | " | ¾ round | ..... | 55.970 | 86.140 | 22.3 | 46.5 | 14″×14″ Ingot | .37 | .85 | |
| " | " | " | ..... | 54.380 | 86.060 | 23.0 | 45.1 | " | | | |
| 6/12 | 4546 | 20×9/16 | .5361 | 44.450 | 75.360 | 24.5 | 44.1 | 180° round ⅝″ | | | |
| 5/26 | " | " | .4562 | 50.200 | 79.130 | 22.0 | 42.0 | 181° round ¾″ | | | |
| " | " | 20×11/16 | .5192 | 48.151 | 79.160 | 24.3 | 45.1 | 180° round ⅝″ | | | |
| 4/28 | " | ¾ round | ..... | 51.060 | 81.280 | 23.8 | 47.1 | 14″×14″ Ingot | | | |
| " | " | " | ..... | 50.800 | 80.320 | 24.8 | 46.2 | " | .34 | .77 | |
| 5/26 | 4547 | 20×⅝ | .4557 | 47.730 | 76.256 | 25.5 | 51.6 | 180° round ½″ | .... | .... | Plate punched soft in spots. |
| " | " | 20×9/16 | .4066 | 49.980 | 76.000 | 25.5 | 51.1 | " | | | |
| 10/3 | " | 20×⅝ | .4719 | 50.070 | 80.000 | 25 0 | 58.3 | | | | |
| 4/28 | " | ¾ round | ..... | 53.770 | 85.620 | 22.6 | 45.7 | 14″×14″ Ingot | .33 | .90 | |
| " | " | " | ..... | 55.400 | 86.040 | 22.3 | 47.2 | " | | | |
| 8/11 | 4651 | 20×⅝ | .4481 | 49.023 | 85.463 | 20.5 | 43.8 | | | | |
| 8/25 | " | " | .4278 | 49.674 | 81.346 | 23.3 | 49.6 | | | | |
| 8/26 | " | 20×¾ | .4604 | 51.324 | 83.615 | 18.5 | 40.8 | | | | |
| 6/11 | " | ¾ round | .... . | 48.810 | 82.180 | 24.4 | 53.2 | 14″×14″ Ingot | .34 | .85 | |
| " | " | " | ..... | 52.050 | 82.620 | 23.8 | 50.2 | " | | | |
| 8/11 | 4665 | 20×9/16 | .5537 | 50.568 | 81.425 | 22.5 | 41.6 | | | | |
| 8/17 | " | " | .4027 | 53.710 | 84.350 | 23.8 | 46.6 | | | | |
| " | " | 20×⅝ | .4484 | 50.235 | 83.868 | 18.0 | 46.4 | | | | |
| " | " | 20×¾ | .4384 | 52.805 | 88.135 | 19.8 | 47.8 | | | | |
| " | " | 20×15/16 | .3371 | 55.681 | 85.170 | 23.8 | 53.3 | | | | |
| 6/11 | " | ¾ round | ..... | 53.730 | 84.740 | 22.9 | 38.6 | 14″×14″ Ingot | .30 | .69 | |
| 6/30 | " | " | ..... | 52.450 | 84.840 | 22.5 | 47.0 | " | | | |
| 8/11 | 4603 | 20×¾ | .5510 | 49.095 | 79.781 | 22.0 | 49.3 | | | | |
| 8/17 | " | " | .5312 | 45.180 | 80.230 | 22.5 | 47.4 | | | | |
| 8/25 | " | 11×⅝ | .4501 | 41.118 | 82.975 | 22.0 | 48.9 | | | | |
| 5/25 | " | ¾ round | ..... | 56.010 | 81.420 | 23.6 | 48.3 | 14″×14″ Ingot | .... | .88 | |
| " | " | " | ..... | 55.360 | 81.340 | 24.3 | 51.2 | " | | | |
| 8/25 | 4618 | 20×11/16 | .4556 | 50.085 | 84.870 | 20.5 | 49.0 | | | | |
| 5/25 | " | ¾ round | ..... | 54.260 | 86.260 | 24.5 | 48.1 | 14″×14″ Ingot | .33 | | |
| " | " | " | ..... | 55.660 | 86.740 | 23.3 | 48.7 | " | | | |
| 8/11 | 4642 | 20×9/16 | .5516 | 47.135 | 82.218 | 16.3 | 34.9 | | | | |
| 8/17 | " | " | .4879 | 51.180 | 87.598 | 22.5 | 37 7 | | | | |
| 6/29 | " | ¾ round | ..... | 50.360 | 80.170 | 25.2 | 51.3 | 14″×14″ Ingot | .34 | .91 | |
| " | " | " | ..... | 50.340 | 80.100 | 25.7 | 51.4 | " | | | |
| 8/11 | 4647 | 16×⅝ | .6115 | 48.082 | 80.874 | 20.0 | 42.0 | | | | |
| 8/17 | " | 20×9/16 | .4965 | 51.620 | 84.830 | 21.3 | 46.1 | | | | |
| " | " | 20×¾ | .4365 | 52.760 | 89.825 | 23.0 | 46.1 | | | | |
| 6/11 | " | ¾ round | ..... | 48.810 | 82.380 | 20.5 | 34.4 | 14″×14″ Ingot | .38 | .93 | |
| " | " | " | ..... | 49.420 | 82.500 | 21.1 | 41.1 | " | | | |
| 6/5 | " | " | ..... | 51.825 | 84.760 | 23.1 | 39.4 | " | | | |
| " | " | " | ..... | 51.740 | 86.040 | 20.7 | 42.4 | " | | | |
| 8/11 | 4668 | 20×⅝ | .4507 | 56.062 | 88.010 | 20.8 | 42.8 | | | | |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/11 | 4668 | ¾ round | ..... | 55.830 | 88.040 | 21.9 | 41.1 | 14″×14″ Ingot | .35 | .80 | |
| 6/30 | " | " | ..... | 54.590 | 88.120 | 22.2 | 48.1 | " | | | |
| 12/9 | 4900 | 11×⅝ | .6162 | 50.265 | 81.100 | 23.0 | 53.4 | 181° round 1″ | | | |
| 10/9 | " | ¾ round | ..... | 59.100 | 83.880 | 22.4 | 42.3 | | | | |
| " | " | " | ..... | 56.940 | 82.200 | 24.1 | 44.5 | | | | |
| 12/9 | 4901 | 12×⅝ | .6200 | 51.950 | 80.646 | 24.5 | 46.4 | 180° round ⅞″ | | | |
| 10/9 | " | ¾ round | ..... | 54.080 | 82.700 | 22.6 | 39.9 | ... ..... | .32 | | |
| " | " | " | .. .. | 54.700 | 82.260 | 21.1 | 37.9 | | | | |
| 10/26 | 4876 | 22×9/16 | .4931 | 46.500 | 81.630 | 22.0 | 50.4 | | | | |
| 7/28 | " | ¾ round | ..... | 55.050 | 80.410 | 22.5 | 40.4 | ... . ..... .. | .37 | | |
| " | " | " | ..... | 55.300 | 80.410 | 22.0 | 39.5 | ..... ..... | " | | |

The following tests were made on plates, the steel slabs for which were obtained at Johnstown and plates rolled at Carnegie's.

In these specimen tests and others given later for ¾″ rounds on ingots cast by Cambria Iron Co., the work done on the test bars was as follows: Ingots generally 18″ square. Heated and rolled to 7″ square blooms. A piece cut from bloom and heated, then hammered to 3″ square billet,—again heated and rolled to ¾″ round bar and then tested without further work.

In the case of the 36″ plates, tests on which immediately follow, the ingots were 28″x12″ and broken down and spread into 36½″x5″ slabs under the hammer.

Considerable difficulty was at first experienced in getting good plates of such unusual size and weight, but extra care in preparation and inspection of the slabs, finally resulted in most satisfactory results.

PLATES USED FOR ALTERNATE *TENSION* AND *COMPRESSION*—OVER ½ INCH THICK.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/9 | 6329 | 36×⅝ | .4156 | 50 530 | 76.516 | 19.5 | 30.0 | | | | |
| " | " | ¾ round | ..... | 47.180 | 76.320 | 21.0 | 42.0 | ..... | .27 | .72 | |
| " | " | " | ..... | 47.450 | 75.910 | 22.1 | 43.3 | ..... .. | ... | ... | E=28.970.000. |
| " | 6436 | 36×⅝ | .4286 | 49.580 | 73.610 | 18.8 | 41.9 | | | | |
| " | " | ¾ round | ..... | 48.610 | 76.250 | 22.1 | 42.4 | .. ..... | .30? | .61 | |
| " | " | " | ..... | 48.960 | 77.940 | 23.1 | 44.6 | ..... ..... | ... | .... | E=29.470.000. |
| " | 6444 | 36×⅝ | .4012 | 48.920 | 72.333 | 17.5 | 40.9 | | | | |
| " | " | ¾ round | ..... | 49.090 | 71.800 | 20.7 | 40.4 | ..... | .25 | .60 | |
| " | " | " | ..... | 49.050 | 72.070 | 22.5 | 38.6 | | | | |
| " | 6466 | 36×⅝ | .4032 | 44.643 | 70.000 | 19.5 | 40.0 | | | | |
| " | " | ¾ round | .. .. | 57.200 | 75.180 | 20.3 | 45.2 | ..... | .22 | .52 | Specimen very cold rolled. |
| " | " | " | .... .. | 57.650 | 75.410 | 21.0 | 45.2 | ..... | .... | .... | " |
| " | 6476 | 36×⅝ | .4265 | 49.355 | 77.490 | 17.0 | 27.2 | | | | |
| " | " | ¾ round | .. .. | 49.210 | 73.810 | 21 8 | 39.4 | ..... | .20 | .67 | |
| " | " | " | ..... | 49.620 | 74.400 | 20.3 | 31.2 | ..... .. | .... | .... | E=29.840.000. |
| 10/18 | 6676 | 36×⅝ | .5810 | 46.690 | 70.220 | 21.3 | 55.4 | | | | |
| 9/13 | " | ¾ round | ..... | 47.420 | 74.920 | 24.3 | 44.1 | 18″×18″ Ingot. | .20 | .83 | |
| " | " | " | ..... | 47.420 | 74.920 | 25.7 | 45.1 | " | .. . | .... | =30.080.000. |

PLATES USED IN *COMPRESSION*—OVER ½ INCH THICK.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 9/9 | 6587 | 36×11/16 | .5985 | 49.575 | 83.520 | 22.0 | 40.6 | | | | |
| 7/30 | " | ¾ round | ..... | 54.730 | 85.760 | 18.1 | 37.9 | .... ..... | .34 | .78 | |
| " | " | " | ..... | 54.500 | 84.650 | 19.6 | 34.7 | ..... ..... | .... | .... | =30.030.000. |

The following tests are for Cambria steel.

In that rolled by the Cambria Company the work done on angles was as follows: Ingots generally 18″ square bloomed down to 7″x7″ for angles under 6″ x4″ and to blooms 7″x8″ for 6″x4″ and 6″x6″ angles.

In angles rolled by the New Jersey Steel and Iron Company, at Trenton, for draw span, the ingots were hammered into billets as follows: 6″x6″ and 6″x4″ angles—billets 9½″x5″; 3½″x3½″ and 3″x3″ angles—billets 4⅞″x4″; 2½″x2½″ angles —billets 3½″ square.

In column headed *ingot* will be noticed the fact, that some tests were made from 7″ square ingots cast for test ingots.

This method of ascertaining the quality of whole heat was soon abandoned as unjust, both to the mill and purchaser, inasmuch as the results obtained did not give a fair judgment.

In column headed *where tested*, the letters have the following significance.

A—tested at shops at Athens.
C— " Cambria mill at Johnstown.
T— " shops at Trenton.
P— " Olsen's at Philadelphia.

## CAMBRIA STEEL, ROLLED BY CAMBRIA COMPANY—ANGLES USED IN *COMPRESSION*.

| Date. | Heat No. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Where Tested. | Size Ingot. | Carbon. | Manganese. | Bending and Modulus. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/21 | 6034 | 3×3×5/16 | .363 | 46.800 | 86.200 | 18.5 | 42.1 | A | ..... | ... | ... | 180°rd ¼″ | ½ silky cup. |
| " | " | 3×3×⅜ | .325 | 44.300 | 83.400 | 19.4 | 43.7 | A | ..... | ... | ... | 180°rd ¾″ | Full " " |
| 3/ | " | ¾ round | .... | 47.260 | 82.560 | 21.7 | 38.6 | C | 7″ sq. | .35 | .52 | | |
| " | " | " | .... | 49.400 | 83.490 | 21.1 | 38.8 | C | " | | | | |
| 4/24 | " | " | .... | 47.900 | 82.520 | 22.1 | 46.8 | C | 18″ sq. | ... | ... | ..... | Flat cup silky. |
| " | " | " | .... | 47.680 | 82.080 | 19.5 | 46.8 | C | " | ... | ... | 28,680,000 | " " " |
| 7/21 | 6038 | 2½×2½×⅜ | .347 | 44.100 | 79.200 | 19.6 | 44.1 | A | ... ... | ... | ... | 180°rd ¾″ | Full " " |
| 3/ | " | ¾ round | .... | 49.790 | 81.360 | 19.8 | 31.7 | C | 7″ sq. | .34 | .49 | | |
| " | " | " | .... | 49.960 | 82.100 | 18.7 | 34.6 | C | " | | | | |
| 4/22 | " | " | .... | 46.430 | 79.320 | 20.5 | 46.3 | C | 18″ sq | ... | ... | ..... | Regular silky. |
| " | " | " | .... | 49.060 | 83.000 | 21.2 | 45.2 | C | " | ... | ... | 30,750,000 | Irreg. " |
| " | " | " | .... | 47.100 | 81.140 | 22.7 | 45.7 | C | " | ... | ... | ..... | Fine cup " |
| " | " | " | .... | 47.100 | 80.680 | 23.2 | 45.6 | C | " | ... | ... | 29,610,000 | Irreg. " |
| 7/21 | 5756 | 4×4×9/16 | .562 | 41.460 | 75.300 | 22.5 | 34.7 | A | ..... | ... | ... | 180°rd 1½″ | Part cup " trace crystals. |
| 7/22 | " | 4×4×11/16 | .555 | 41.100 | 78.100 | 24.4 | 50.3 | A | ..... | ... | ... | 180°rd 1¾″ | Part cup silky trace crystals. |
| 3/ | " | ¾ round | .... | 49.350 | 78.320 | 20.6 | 34.9 | C | 7″ sq. | .33 | .53 | | |
| " | " | " | .... | 49.350 | 78.090 | 20.4 | 39.2 | C | " | | | | |
| 4/23 | " | " | .... | 48.030 | 81.600 | 19.7 | 34.2 | C | 18″ sq. | ... | ... | ..... | ½ cup silky. |
| " | " | " | .... | 47.800 | 81.180 | 22.0 | 34.5 | C | " | ... | ... | 29,680,000 | " " " |
| 7/21 | 6007 | 6×4×½ | .485 | 41.600 | 82.500 | 23.1 | 46.9 | A | ..... | ... | ... | ..... | Part " " trace crystals. |
| " | " | " | .491 | 43.800 | 38.900 | 22.4 | 46.2 | A | ..... | ... | ... | ..... | ½ cup silky. |
| " | " | 6×4×9/16 | .597 | 42.900 | 85.600+ | .... | .... | A | ..... | ... | ... | ..... | Not broken. |
| 3/ | " | ¾ round | ... | 47.720 | 85.310 | 20.0 | 36.8 | C | 7″ sq. | .36 | .61 | | |
| " | " | " | .... | 50.180 | 85.060 | 18.7 | 35.7 | C | " | | | | |
| 4/22 | " | " | .... | 49.560 | 87.660 | 20.9 | 45.5 | C | 18″ sq. | ... | ... | ..... | Flat cup silky |
| " | " | " | .... | 49.260 | 86.950 | 21.6 | 45.8 | C | " | ... | ... | 28,910,000 | " " " |
| 5/4 | " | " | .... | 45.420 | 83.530 | 22.5 | 43.5 | C | " | ... | ... | ..... | Irreg. silky some crystals. |
| " | " | " | .. | 46.470 | 83.740 | 21.7 | 42.3 | C | " | ... | ... | ..... | Irreg. silky. |
| 10/22 | 6563 | 6×4×70 lbs L | .595 | 45.500 | 84.030+ | 9. | 8.4 | A | ..... | ... | ... | 180° rd 2″ | Could not break. |

## TESTS OF ANGLES USED IN *COMPRESSION.—Continued.*

| Date. | Heat No. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch: | Elongation, % in 8″. | Reduction of Area, %. | Where Tested. | Size Ingot. | Carbon. | Manganese. | Bending and Modulus. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/23 | 6563 | ¾ round | .... | 58.170 | 87.830 | 18.8 | 39.2 | C | 18″ sq. | .32 | .89 | ..... | Irreg. silky. |
| " | " | " | .... | 58.170 | 88.280 | 20.3 | 40.6 | C | " | ... | ... | 29.450.000 | " " |
| 10/22 | 6532 | 4×4×53.5lb. L | .503 | 38.100 | 73.300 | 26.4 | 53.7 | A | ..... | ... | ... | 180° rd 2″ | ½ cup silky. |
| 7/13 | " | ¾ round | .... | 51.250 | 79.540 | 22.5 | 41.9 | C | ..... | .29 | .71 | ..... | Silky. |
| " | " | " | .... | 52.410 | 78.840 | 23 8 | 44.9 | C | ..... | ... | ... | 29.440.000 | " |
| 10/22 | 6047 | 4×4×7/16 L | .430 | 43.500 | 87.400 | 19.0 | 43.5 | A | ..... | ... | ... | 180° rd 1″ | ¾ cup silky. |
| 3/ | " | ¾ round | .... | 51.360 | 85.760 | 20.7 | 33.8 | C | 7″ sq. | .37 | .81 | | |
| " | " | " | ... | 52.320 | 86.460 | 19.3 | 34.0 | C | " | | | | |
| 4/22 | " | " | .... | 51.520 | 85.550 | 22.2 | 43.8 | C | 18″ sq. | ... | ... | ..... | Very irreg. |
| " | " | " | .... | 51.780 | 86.000 | 22.2 | 42.8 | C | " | ... | ... | 31.200.000 | Flat cup silky. |
| 10/22 | 6525 | 3×3×⅜ L | .373 | 47.900 | 85.000 | 21.3 | 31.7 | A | ..... | ... | .. | ..... | ¾ " " |
| 7/10 | " | ¾ round | .... | 53.510 | 86.480 | 19.0 | 23.8 | C | 18″ sq. | .36 | .89 | ..... | Crystalline & laminated. |
| " | " | " | .... | 53.740 | 87.180 | 21.6 | 33.8 | C | " | ... | ... | 30.000.000 | Crystalline & laminated. |
| 10/22 | 6560 | 2½×2½×5/16 L | .323 | 47.300 | 81.600 | 21.3 | 43.0 | A | ..... | ... | ... | 180°rd ½″ | ½ cup silky. |
| " | " | " | .314 | 47.400 | 84.400 | 20 3 | 36.6 | A | ..... | ... | ... | 170°rd ½″ | ¾ " " |
| " | " | 2½×2½×20 lbs | .402 | 46.000 | 83.600 | 21.3 | 34.1 | A | ..... | ... | ... | 180°rd ¾″ | ½ " " |
| " | " | " | .386 | 45.800 | 84.200 | 22.5 | 43.8 | A | ..... | ... | ... | 180°rd ½″ | ¾ " " |
| 7/22 | " | ¾ round | .... | 53.600 | 85.800 | 22.3 | 42.1 | C | 18″ sq. | .31 | .87 | ..... | Perf. cup " |
| " | " | " | .... | 53.450 | 84.400 | 21.0 | 42.2 | C | " | ... | ... | 31.520.000 | " " " |
| 7/21 | 6266 | 2½×2½×5/16 L | .295 | 49.500 | 81.300 | 17.5 | 40.0 | A | ... ... | ... | ... | 180° rd 1″ | Rag'd " " |
| 4/23 | " | ¾ round | .... | 47.870 | 80.480 | 22 0 | 48.0 | C | 18″ sq. | .28 | .85 | | |
| " | " | " | .... | 47.870 | 80.000 | 23.0 | 47.6 | C | " | ... | ... | 29.440.000 | Perf. " " |

## CAMBRIA STEEL, ROLLED AT TRENTON—ANGLES USED IN COMPRESSION.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Where Tested. | Ingot. | Carbon by Cambria. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 6780 | 3½″L28.9 lbs | .... | 46.630 | 84.610 | 16.0 | 18.8 | T | ..... | .33 | .... | 80% cryst-sq. |
| | " | " " | .... | 46.960 | 83.750 | 19.5 | 40.0 | T | ..... | .30 | ... | Silky cupped. |
| | " | " " | .... | 47.210 | 82.770 | 23.5 | 51.7 | C | ..... | .32 | .... | " " |
| | " | " " | .... | 46.160 | 81.940 | 22.7 | 49.7 | C | ..... | ... | .... | " faint cryst. |
| | " | " " | .... | 47.050 | 85.280 | 18.2 | 32.4 | T | ..... ... | .30 | .... | " angular. |
| | " | " " | .... | 49.140 | 83.390 | 20.0 | 49.4 | C | ..... | .31 | .... | " |
| | " | " " | ... | 46.020 | 81.580 | 22.1 | 47.5 | C | ..... | .... | .... | " |
| | " | 2½″L14 2 lbs | .... | 45.170 | 80.000 | 18.0 | .... | T | ..... ..... | .... | .... | " angular. |
| | " | " " | .... | 46.200 | 82.500 | 18.7 | 44.3 | T | ..... | .... | .... | " cupped. |
| 9/29 | " | ¾ round | .... | 54.060 | 84.880 | 21.2 | 38.1 | C | 7″sq. ingot | .32 | .82 | " ½ cup. |
| " | " | " | .... | 54.060 | 84.880 | 21.9 | 38.1 | C | " | .... | .... | " irregular. E=30.160.000. |
| | 6782 | 2½″L-14.2 lbs | .... | 46.860 | 82.830 | 18.7 | 41.5 | T | ..... | ... | .... | Silky cupped. |
| 9/30 | " | ¾ round | .... | 50.920 | 83.540 | 21.6 | 44.0 | C | 14″sq. ingot | .32 | .66 | " irregular. |
| " | " | " | ... | 51.510 | 82.870 | 21.8 | 43.8 | C | " | .... | .... | " " E=30.380.000. |
| 10/26 | 6781 | 3½″L-27 lbs | .400 | 41.250 | 78.500 | 23.5 | 43.8 | T | ..... | .... | .... | Silky cupped. |
| " | " | " | .389 | 43.950 | 80.840 | 21.9 | 44.8 | T | ... ..... | .... | .... | " " |
| 9/30 | " | ¾ round | .... | 49.800 | 80.580 | 22.1 | 44.2 | C | 14″sq. ingot | .32 | .67 | " " |
| " | " | " | .... | 49.800 | 80.360 | 21.9 | 44.2 | C | " | .... | .... | " ½ cup. E=29.-690.000. |
| 11/28 | 6788 | 3″L-25 lbs | .362 | 45.000 | 82.870 | 18.8 | 43.4 | P | ..... | .... | .... | Silky square. |
| " | " | " " | .368 | 46.640 | 83.460 | 17.7 | 37.8 | P | | | | |
| " | " | " " | .370 | 45.940 | 83.790 | 18.0 | 34.6 | P | ..... | .... | .... | " cupped. |

## CAMBRIA STEEL, ROLLED AT TRENTON—ANGLES USED IN COMPRESSION ACCEPTED ON INGOT TESTS AND REJECTED ON FINISHED TESTS.

| Test Mark. | Heat No. | Material. | Where Tested. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Carbon by Salom, Phila. | Carbon by Cambria. | Remarks as to Fracture. |
|---|---|---|---|---|---|---|---|---|---|---|
| F | 6741 | 3½″ L 40 lbs | T | 43.720 | 77.200 | 21.8 | 45.9 | .35 | .31 | Silky cupped. |
| G | " | " " | T | 43.650 | 78.500 | 18.7 | 41.5 | .38 | .32 | " square. |
| 1 | " | " " | C | 42.510 | 76.150 | 22.9 | 49.8 | ..... | .30 | " 100%. |
| 1x | " | " " | C | 42.050 | 74.330 | 21.5 | 42.5 | ..... | ..... | " faintly cryst. |
| C | " | " 37.8 lbs | T | 41.100 | 70.090 | 22.5 | 40.0 | .25-.245 | .20 | " at 45°. |
| I | " | " 40 lbs | T | 42.880 | 83.690 | 18.7 | 29.0 | .47-.49 | .38-.43-.39 | Crystalline fine. |
| 3 | " | " 37.8 lbs | C | 42.300 | 71.970 | 23.2 | 45.2 | ..... | .27 | Silky partly. |
| 3x | " | " " | C | 44.850 | 82.920 | 19.0 | 26.4 | ..... | ..... | Square-cryst. |
| * | " | " 30 lbs | T | 41.260 | 78.030 | 19.5 | 38.2 | ..... | ..... | Silky cupped. |
| 1 | " | " 30.5 lbs | T | 44.420 | 75.390 | 24.2 | 34.8 | ..... | ..... | " " |
| † | " | " " | T | 40.080 | 73.780 | 22.6 | 45.1 | .31-.31 | .30 | " " |
| 7 | " | " 30 lbs | C | 44.090 | 80.740 | 21.2 | 38.5 | ..... | .42 | " ½ cryst ½. |
| 7x | " | " " | C | 45.570 | 80.710 | 22.3 | 36.6 | ..... | ..... | " ¼ " ¾. |
| 8 | " | " " | C | 41.880 | 79.820 | 20.6 | 43.2 | ..... | .36 | " ½ " ½. |
| 8x | " | " " | C | 46.340 | 81.470 | 19.5 | 47.4 | ..... | ..... | " 100%. |
| | " | ¾ round | C | 48.560 | 78.630 | 21.8 | 39.9 | ..... | .28 | |
| | " | " | C | 48.560 | 78.630 | 23.2 | 44.1 | | | |
| H | 6740 | 3½″ L 37.8 lbs | T | 45.000 | 80.000 | 18.7 | 34.8 | .34 | .30 | Silky cupped. |
| E | " | " " | T | 45.610 | 81.570 | 17.1 | 35.6 | .40 | .35 | " $^{9}/_{10}$-angular. |
| 4 | " | " " | C | 44.150 | 79.090 | 16.4 | 22.1 | ..... | .34 | Square crystalline. |
| 4x | " | " " | C | 48.120 | 90.610 | 13.7 | 15.9 | ..... | ..... | " " 100%. |
| ‡ | " | 3″ L 30.5 lbs | T | 46.630 | 84.750 | 16.3 | 22.4 | .45-.49 | .47-.32-.465 | " " |
| 6 | " | " | C | 44.200 | 86.360 | 15.0 | 17.1 | ..... | .35 | " " " |
| 6x | " | " | C | 43.750 | 77.850 | 19.3 | 38.0 | ..... | ..... | Silky. |
| | " | ¾ round | C | 49.720 | 82.100 | 21.7 | 41.0 | ..... | .34 | |
| | " | " | C | 50.880 | 81.640 | 21.0 | 41.0 | | | |

## CAMBRIA STEEL, ROLLED BY CAMBRIA—ANGLES USED IN TENSION, OR ALTERNATE TENSION AND COMPRESSION.

| Date. | Heat No. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Where Tested. | Size Ingot. | Carbon. | Manganese. | Bending and Modulus. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/22 | 6319 | 2½×2½×⅜ | .425 | 42.800 | 70.100 | 21.9 | 48.2 | A | ..... | .. | ... | 180° round ½″ | ½ cup silky. |
| 5/9 | " | ¾ round | .... | 44.970 | 71.350 | 22.5 | 41.4 | C | 18″ sq. | .20 | .83 | ..... | Irregular silky. |
| " | " | " | ... | 46.100 | 71.800 | 24.8 | 43.5 | C | " | ... | ... | 29.360.000 | " |
| 7/21 | 6332 | 6×4×69 lbs | .541 | 36.600 | 69.600 | 24.3 | 50.8 | A | ..... | ... | ... | 180° round 2″ | ½ cup silky. |
| " | " | " | .546 | 38.600 | 69.800 | 23.1 | 42.5 | A | ..... | ... | ... | 180° round 2″ | Full cup silky. |
| 5/26 | " | ¾ round | .... | 46.110 | 74.550 | 22.1 | 42.5 | C | 18″ sq. | .26 | .94 | ..... | ½ cup silky. |
| " | " | " | .... | 45.740 | 73.960 | 24.1 | 44.9 | C | " | ... | ... | 28.900.000 | " |
| 7/22 | 6388 | 3×3×⅜ | .343 | 41.400 | 72.300 | 24.4 | 53.3 | A | ..... | .. | ... | 180° round ¾ | " |
| " | " | 3×3×5/16 | .344 | 42.400 | 73.500 | 22.5 | 52.0 | A | ..... | ... | ... | 180° round ½ | " |
| 5/28 | " | ¾ round | .... | 43.240 | 71.360 | 25.4 | 49.7 | C | 18″ sq. | .23 | .73 | ..... | " |
| " | " | " | .... | 43.110 | 70.920 | 24.7 | 50.7 | C | " | ... | ... | ..... | Deep cup silky. |
| 7/22 | 6259 | 3×3×21.5 lbs | .349 | 42.400 | 73.300 | 21.0 | 47.5 | A | ..... | ... | ... | 180° round 1″ | ½ cup silky. |
| 4/18 | " | ¾ round | .... | 43.710 | 72.700 | 20.3 | 41.3 | C | 18″ sq. | .24 | .54 | ..... | Silky. |
| " | " | " | ... | 45.200 | 72.500 | 21.4 | 40.4 | C | " | ... | ... | ..... | " |
| 7/22 | 6270 | 2½×2½×5/16 | .308 | 45.400 | 76.400 | 21.9 | 52.9 | A | ..... | ... | ... | 180° round 1″ | Perf. cup silky. |
| 4/23 | " | ¾ round | ... | 45.100 | 73.460 | 24.1 | 44.8 | C | 18″ sq. | .27 | .86 | ..... | ½ cup silky. |
| " | " | " | .... | 44.870 | 73.000 | 23.4 | 45.8 | C | " | ... | ... | 28.830.000 | " |
| 7/22 | 6272 | 5×3½×⅜ | .385 | 42.600 | 73.000 | 24.4 | 51.9 | A | ..... | ... | ... | 180° round ¾″ | Full cup silky. |
| 4/23 | " | ¾ round | .... | 45.560 | 72.850 | 23.5 | 43.0 | C | 18″ sq. | .24 | .64 | ..... | ½ cup silky. |
| " | " | " | .... | 45.330 | 72.850 | 23.8 | 39.9 | C | " | ... | ... | 30.830.000 | Silky. |
| 10/22 | 6571 | 6×4×69 lbs | .544 | 39.900 | 71.200 | 25.4 | 53.1 | A | ..... | ... | ... | 180° round 1½ | ¾ cup silky. |
| " | " | 2½×2½×30 lbs. | .470 | 40.200 | 74.700 | 25.5 | 51.7 | A | ..... | ... | ... | 180° round 1″ | |
| " | " | 2½×2½×30 lbs. | .465 | 43.200 | 75.600 | 26.4 | 47.7 | A | ..... | ... | ... | 180° round 1½″ | ¾ cup silky. |
| 7/27 | " | ¾ round | .... | 49.390 | 76.690 | 23.8 | 47.9 | C | 18″ sq. | .24 | .94 | ..... | Irreg. silky. |
| " | " | " | .... | 50.960 | 76.890 | 23.2 | 44.9 | C | " | ... | ... | 30.860.000 | " |
| 10/22 | 6555 | 4×4×53.5 lbs | .518 | 39.000 | 77.200 | 23.4 | 48.8 | A | ..... | ... | ... | 180° round 1½ | ¾ cup silky. |
| " | " | 4×4×33.2 lbs | .404 | 41.500 | 78.200 | 24.0 | 46.5 | A | ..... | ... | ... | 180° round 1″ | ½ cup irreg. |
| 7/20 | " | ¾ round | .... | 47.980 | 73.340 | 24.0 | 50.1 | C | 18″ sq. | .20 | .88 | ..... | Perf. cup silky. |
| " | " | " | .... | 47.290 | 70.580 | 24.0 | 49.1 | C | " | ... | ... | 28.830.000 | Imp. cup silky. |
| 10/22 | 6344 | 3×3×11/16 L. | .510 | 43.500 | 77.400 | 24.0 | 41.8 | A | ..... | ... | ... | 180° round 1½″ | ½ cup irreg. |
| " | " | " | .502 | 40.100 | 76.000 | 23.9 | 44.4 | A | ..... | ... | ... | 180° round ½ | " |
| 5/13 | " | ¾ round | .... | 45.570 | 75.310 | 23.2 | 38.6 | C | 18″ sq. | .24 | .41 | ..... | Irregular silky. |
| " | " | " | .... | 44.610 | 73.500 | 23.2 | 40.9 | C | " | ... | ... | 28.900.000 | " |
| 10/22 | 6570 | 3×3×7/16 L. | .406 | 43.100 | 74.600 | 20.1 | 40.9 | A | ..... | ... | ... | 180° round 1″ | ½ cup silky. |
| " | " | " | .417 | 42.200 | 73.900 | 23.3 | 53.9 | A | ..... | ... | ... | 180° round 1″ | ¾ cup silky. |
| 7/30 | " | ¾ round | .... | 46.920 | 71.790 | 23.1 | 46.4 | C | 18″ sq. | .22 | .63 | ..... | ½ cup silky. |
| " | " | " | .... | 45.750 | 70.940 | 24.3 | 48.5 | C | " | ... | ... | 30.260.000 | " |
| 10/22 | 6565 | 2½×2½×5/16 | .313 | 43.200 | 71.200 | 19.8 | 52.1 | A | ..... | ... | ... | 180° round ½″ | ½ cup irreg. |
| " | " | 2½×2½×5/16 | .309 | 42.900 | 70.600 | 25.6 | 50.0 | A | ..... | ... | ... | 180° round ¼″ | ⅔ cup silky. |
| 7/23 | " | ¾ round | .... | 46.380 | 70.740 | 27.0 | 53.5 | C | 18″ sq. | .18 | .90 | ..... | Deep cup silky. |
| " | " | " | .... | 47.080 | 70.260 | 27.5 | 54.4 | C | " | ... | ... | 29.920.000 | Irregular silky. |
| | 6730 | 3½″ L 40 lbs | .... | 41.340 | 70.940 | 26.0 | 47.6 | C | ..... | .29 | | | |
| | " | " " " | .... | 40.710 | 70.650 | 22.3 | 45.0 | C | | | | | |

## CAMBRIA STEEL, ROLLED AT TRENTON—ANGLES USED IN *TENSION.*

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit: lbs. per Sq. Inch. | Ultimate Strength: Pounds per Sq. In. | Elongation, % in 8″. | Reduction of Area, %. | Tested at. | Size Ingot. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 11/14 | " | 3″ L 17.7 lbs | .341 | 42.810 | 73.630 | 20.6 | 41.7 | T | ... ... | ... | ... | Silky at 45°. |
| " | " | " " " | .352 | 42.040 | 70.020 | 20.3 | 44.4 | T | ..... | ... | ... | " cupped. |
| " | " | " " 25.3 | .447 | 40.940 | 70.800 | 23.5 | 42.6 | T | ..... | ... | ... | " at 45°. |
| 9/15 | " | ¾ round | .... | 45.620 | 72 550 | 22.5 | 51.5 | C | 14″ sq. Ingot | .28 | .55 | " irregular. |
| | " | " " | .... | 47.000 | 72.550 | 25.0 | 52.4 | C | " | ... | ... | Silky irregular E=30.600.000. |
| 11/14 | 6775 | 3″ L-33.2 lbs | .469 | 43.710 | 76.870 | 21.8 | 41.0 | T | ..... | ... | . | Silky cupped. |
| 9/29 | " | ¾ round | .... | 48.180 | 77.530 | 22.0 | 45.5 | C | 7″ sq. Ingot | .25 | .95 | " partial cup. |
| " | " | " | .... | 49.790 | 77.300 | 24.2 | 44 5 | C | " | ... | ... | Silky partial cup. E=30.080.000. |
| 11/14 | 6776 | 3″ L 33.2 lbs | .478 | 42.050 | 72.800 | 25.0 | 46.7 | T | ..... | ... | ... | Silky cup. |
| 9/29 | " | ¾ round | .... | 50.470 | 76.830 | 24.3 | 45.5 | C | 7″ sq. Ingot | .22 | .90 | " partial cup. |
| " | " | " | .... | 50.470 | 76.620 | 25.5 | 46.5 | C | " | ... | ... | Silky partial cup. E=30.590.000. |
| 11/14 | 6785 | 6″×6″L×⅝″ | .461 | 41.560 | 70.900 | 23.8 | 44.2 | T | ..... | ... | ... | Silky cupped. |
| " | " | " | .488 | 42.820 | 71.920 | 23.8 | 35.3 | T | ..... | ... | ... | " $^{9}/_{10}$-angular. |
| " | " | 6″×4″L×¾″ | .530 | 41.700 | 70.560 | 25.0 | 45.1 | T | ..... | ... | ... | Silky cupped. |
| " | " | " " | .547 | 40.210 | 70.200 | 24.0 | 39.0 | T | ..... | ... | ... | " " |
| 9/29 | " | ¾ round | .... | 47.160 | 72.750 | 25.1 | 48.6 | C | 14″ sq. Ingot | .21 | .59 | " cup. |
| " | " | " | .... | 47.160 | 71.850 | 24.2 | 49.5 | C | " | ... | ... | Silky irregular. E=30.460.000. |
| 11/14 | 6786 | 3″ L 28.0 lbs | .507 | 42.010 | 71.990 | 20.8 | 49.3 | T | ..... | ... | ... | Silky-¾ angular. |
| 10/1 | " | ¾ round | .... | 48.920 | 72.130 | 22.1 | 42.9 | C | 14″ sq. Ingot | .18 | .83 | " irregular. |
| " | " | " | .... | 49.610 | 73.710 | 23.9 | 41.9 | C | " | ... | ... | Silk irregular. E=29.600.000. |
| 11/14 | 6795 | 3″ L-28 lbs | .515 | 41.300 | 70.870 | 18.8 | 41.6 | T | ..... | ... | .. | Silky cupped. |
| 10/3 | " | ¾ round | .... | 50.260 | 75.510 | 21.7 | 35.0 | C | 14″ sq. Ingot | .24 | .7½ | " irregular. |
| " | " | " | .... | 50.970 | 75.510 | 23.8 | 40.0 | C | " | ... | .. | Silky cupped. E=29.690.000. |

In the following tests on rivet steel, the first four were made at mill on ¾ round, from ingot.

The other tests were made at shop at Athens on the regular rivet bars. The 3″ square billets were rolled into rivet material at Pencoyd Mills. All the rivets tested in this lot came from the heats 4635 and 4637, but as rivet bars had no heat numbers stamped on them they could not be identified closely.

## PENNA. STEEL CO.'S STEEL, ROLLED AT PENCOYD—STEEL RIVETS.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6/5 | 4635 | ¾ round | .500 | 46.370 | 61.310 | 28.5 | 54.3 | 18″ sq. Ingot. | .10 | .40 | ½ cup silky. |
| " | " | " | .495 | 45.700 | 61.340 | 26.7 | 55.1 | " | | | " |
| " | 4637 | " | .525 | 44.550 | 61.650 | 27.5 | 48.7 | " | .11 | .44 | " |
| " | " | " | .525 | 43.940 | 61.650 | 26.4 | 48.8 | " | | | " |
| 6/24 | .... | ⅞ round | .594 | 24.300? | 56.230 | 33.4 | 64.3 | | | | |
| " | ... | " | .603 | 27.200 | 59.200 | 30.0 | 62.0 | | | | |
| 6/26 | ... | ¾ round | .443 | 25.300? | 59.600 | 30.0 | 64.1 | Specimen not treated. | | | |
| " | .... | " | .443 | 28.200 | 59.140 | 31.3 | 64.1 | " " | | | |
| 6/27 | .... | " | .447 | 38.500 | 59.060 | 28.8 | 62.9 | " reheated then air cooled. | | | |
| " | ... | " | .445 | 34.800 | 57.900 | 30.0 | 64.3 | " " " | | | |
| 6/29 | .... | " | .443 | 26.840 | 55.080 | 29.6 | 60.9 | " annealed. | | | |
| " | .... | " | .441 | 26.500 | 54.400 | 30.6 | 63.9 | " " | | | |
| 6/26 | .... | " | .438 | 29.200 | 73.500 | 15.6 | 62.1 | " heated bright red then quenched in water. | | | |
| " | .... | " | .439 | 28.000 | 74.260 | .. | .... | " " " | | | broke in jaw. |
| 6/27 | .... | " | .443 | 45.100 | 73.100 | 15.3 | 59.1 | " " " | | | |
| " | .... | " | .438 | 52.050 | 75.200 | .... | 63.7 | " " " | | | " |

BENDING TESTS.

⅞″ round Rods—Cherry red—then quenched in water; nicked and bent broke at 60°; without nicking bent double around ½″ round, no crack.

⅞″ round Rods—Bright red—then quenched, bent double around ½″ round, no crack.

¾″ round—2 pieces—Bright red—then quenched, bent double around ½″ round, no crack.

¾″ round—2 pieces—Bright red—then air cooled, bent double around ½″ round, no crack.

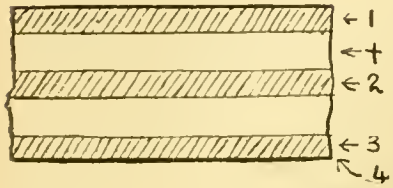
⅞″ round—3 pieces—Bright red—then air cooled, bent double around ¼″ round, no crack.

When tested with file no perceptible difference detected between quenched and air cooled.

## PENNA. STEEL CO.'S STEEL, ROLLED AT PENCOYD—STEEL RIVETS.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8″. | Reduction of Area, %. | Bending and Ingot Notes. | Carbon. | Manganese. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 8/6 | 4767 | ¾ round | .531 | 42.220 | 60.610 | 27.5 | 49.7 | Ingot tests made at Steelton. | .09 | .35 | |
| " | " | " | .513 | 42.380 | 60.500 | 28.3 | 53.3 | | | | |
| " | 4769 | " | .523 | 40.870 | 62.000 | 27.5 | 51.1 | | .08 | .43 | |
| " | " | " | .512 | 41.650 | 61.840 | 27.8 | 53.2 | | | | |
| 8/17 | 4775 | " | .496 | 41.270 | 63.190 | 28.5 | 56.6 | | .10 | .42 | |
| " | " | " | .500 | 40.750 | 63.360 | 28.5 | 55.6 | | | | |
| " | 4777 | " | .497 | 39.260 | 60.460 | 29.3 | 55.9 | | .09 | .35 | |
| " | " | " | .505 | 38.970 | 59.670 | 27.8 | 54.9 | | | | |
| " | 4779 | " | .505 | 38.910 | 59.750 | 30.5 | 55.2 | | .09 | .37 | |
| " | " | " | .497 | 38.730 | 59.530 | 28.8 | 56.1 | | | | |
| " | 4782 | " | .475 | 37.780 | 58.590 | 28.8 | 60.1 | | .10 | .48 | |
| " | " | " | .498 | 38.250 | 58.980 | 28.8 | 56.1 | | | | |
| 8/ | .... | " | .454 | 39.000 | 61.200 | 28.4 | 63.4 | Rivet bar tests made at Athens. | .... | .... | Full cup silky. |
| 9/21 | .... | ⅞ " | .605 | 38.800 | 59.900 | 30.8 | 66.3 | | ... | .... | ¾ " |
| " | .... | " | .605 | 37.300 | 58.200 | 30.0 | 67.6 | | ... | .... | Full " |
| " | .... | " | .600 | 35.500 | 58.400 | 28.4 | 68.5 | | .... | .... | ⅞ " |
| " | .... | " | .597 | 36.300 | 58.800 | 32.6 | 67.2 | | .... | .... | ¾ " |

## CAMBRIA STEEL, ROLLED BY CAMBRIA—SPECIMEN TESTS ON EYEBAR STEEL.




No. 1. Edge of bar, ⅛″ skin planed off.
" +. Between edge and center.
" 2. Centre of bar.
" 3. Edge of bar, ⅛″ skin planed off.
" 4. Edge of bar, skin left on.

In all specimens width equals thickness of bar, except those marked *, which were turned up round. The four tests from Heat No. 6391, marked *, were from same bar as those immediately preceding.

| Date. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit per Sq. Inch. | Ultimate Strength per Sq. Inch. | Elongation, % in 8″ | Reduction of Area, %. | Ingot and Bending. | Carbon. | Manganese. | Mark. | Modulus and Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7/2 | 6324 | 3×¾ | .622 | 41.480 | 75.240 | 23.7 | 34.7 | ... | .... | .... | No. 2 | 1/3 Crystalline. |
| " | " | " | .628 | 41.080 | 75.160 | 26.1 | 46.0 | ... | .... | .. | No. 1 | Silky. |
| ..... | " | ¾ round | ... | 47.860 | 75.910 | 23.1 | 43.3 | 18″×18″ Ingot | .20 | .66 | | |
| ..... | " | " | .... | 47.720 | 75.700 | 22.5 | 41.4 | " | .... | .... | ..... | E=30.080.000 |
| 7/2 | 6325 | 4×13/16 | .611 | 41.700 | 71.800 | 23.7 | 39.6 | ... | .... | .... | No. 1 | ¼ Crystalline. |
| " | " | " | .608 | 40.800 | 70.560 | 23.1 | 45.6 | ... | .... | .... | No. 2 | Silky. |
| 5/11 | " | ¾ round | .... | 47.590 | 75.500 | 22.5 | 40.6 | ... | .20 | .60 | ..... | |
| 5/11 | " | " | .... | 47.350 | 75.270 | 23.0 | 38.5 | ... | .... | ... | ..... | E=30.260.000 |
| 7/2 | 6339 | 5×31/32 | .484 | 40.700 | 75.200 | 24.4 | 47.7 | 180° round 2″ | .... | ... | No. 1 | Silky. |
| " | " | " | .479 | 43.800 | 72.900 | 25.6 | 41.1 | ... | .... | .. | No. 2 | ⅛ Fine crystal specks. |
| 5/16 | " | ¾ round | .... | 45.820 | 73.400 | 23.4 | 39.3 | 18″×18″ Ingot | .21 | .68 | | |
| " | " | " | .... | 45.330 | 72.610 | 22.7 | 38.7 | " | .... | .... | ..... | ....E=29.600.000 |
| 7/3 | 6341 | 5×13/16 | .591 | 42.100 | 79.300 | 20.9 | 22.8 | 140°-broke.... | .... | .... | No. 1 | ⅞ ⅛ Silky. Crystalline. |
| " | " | " | .591 | 42.600 | 79.700 | 19.6 | 24.3 | ... | .... | .... | No. 2 | " |
| 5/16 | " | ¾ round | .... | 48.890 | 78.420 | 22.0 | 39.3 | 18″×18″ Ingot | .22 | .87 | | |
| " | " | " | .... | 49.160 | 78.850 | 22.4 | 37.7 | " | ... | .... | ..... | E=31.410.000 |
| 7/1 | 6290 | 6×1 | .659 | 38.550 | 69.730 | 26.3 | 65.5 | 180° round 1½″ | .... | .... | No. 1 | Silky. |
| " | " | " | .706 | 36.800 | 65.650 | 25.1 | 56.1 | ... | .... | .... | No. 4 | " |
| " | " | " | .611 | 38 950 | 76.760 | 26.3 | 52.5 | ... | .... | .... | No. 2 | " |
| 4/29 | " | ¾ round | .... | 43.710 | 72.700 | 24.8 | 45.2 | 18″×18″ Ingot | | | | |
| " | " | " | .... | 45.200 | 72.500 | 25.1 | 45.2 | " | .... | .... | ..... | E=29.290.000 |
| 7/2 | 6391 | 7×1⅜ | .592 | 39.870 | 77.950 | 13.3 | 13.3 | 140°-broke.... | .... | .... | No. 2 | Broke near jaw. Completely crystalline. |
| " | " | " | .594 | 39.900 | 76.200 | 12.0 | 11.8 | ... | .... | .... | No. 2 | " |
| " | " | " | .632 | 37.300 | 69.940 | 24.4 | 38.6 | ... | ... | .... | No. 1 | 60% Crystalline. |
| " | " | " | .630 | 36.800 | 69.290 | 24.3 | 44.9 | ... | .... | .... | No. 3 | 30% " |
| 7/7 | 6391 | 7×1⅜ | *.444 | 41.440 | 77.200 | 23.1 | 48.4 | 150°-broke | .... | .. . | No. 1 | Fairly silky. |
| " | " | " | *.448 | 40.620 | 76.700 | 23.0 | 43 1 | ... | ... | .... | No. 3 | " |
| " | " | " | *.415 | 38.200 | 72.900 | 22.3 | 36.8 | ... | .... | .... | No. 2 | ¼ Crystalline. |
| " | " | " | *.445 | 38.400 | 72.990 | 21.5 | 36.8 | ... | .... | .... | No. 2 | 30% " |
| 6/1 | " | ¾ round | ..... | 46.630 | 77.630 | 22.7 | 42.2 | 18″×18″ Ingot | .27 | .55 | ..... | |
| " | " | " | ..... | 46.170 | 76.940 | 21.7 | 43.2 | " | .... | .... | ..... | E=28.750.000. |
| 6/23 | 6173 | " | ..... | 49.620 | 76.990 | 21.2 | 43.9 | ... | .26 | .57 | ..... | |
| " | " | " | ..... | 49.730 | 75.970 | 23.6 | 44.8 | ... | .... | ... | ..... | E=30.170.000. |
| 7/29 | 6589 | " | ..... | 47.910 | 73.670 | 23.3 | 44.8 | ... | .23 | .77 | ..... | |
| 7/30 | " | " | .. ... | 48.310 | 74.510 | 22.8 | 46.4 | ... | .... | .... | ..... | |
| 6/19 | 6469 | " | ..... | 49.560 | 74.780 | 20.9 | 35.1 | 18″×18″ Ingot | .25 | .58 | ..... | |
| " | " | " | ..... | 50.470 | 74.560 | 20.4 | 35.1 | " | .... | .... | ..... | E=29.140.000. |
| 10/8 | 6794 | " | ..... | 46.310 | 69.750 | 23.9 | 44.5 | ... | .19 | .50 | .. ... | |
| " | " | " | ..... | 46.800 | 69.500 | 21.8 | 43.5 | ... | .... | ... | ..... | E=29.840.000. |
| 10/2 | 6790 | " | ..... | 49.160 | 74 180 | 22.1 | 42.9 | ... | .23 | .84 | ..... | |
| " | " | " | ..... | 49.160 | 75.100 | 21.5 | 42.9 | ... | .... | .... | ..... | E=29.840.000. |
| 6/18 | 6461 | " | ..... | 48.670 | 69.420 | 24.0 | 46.2 | 18″×18″ Ingot | .20 | .57 | ..... | |
| " | " | " | ..... | 46.820 | 69.190 | 25.9 | 48.1 | " | .... | .... | ..... | E=30.750.000. |

SPECIMEN TESTS ON EYEBAR STEEL.—*Continued.*

| Date. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit per Sq. inch. | Ultimate Strength per Sq. inch. | Elongation, % in 8.″ | Reduction of Area, %. | Ingot and Bending. | Carbon. | Manganese. | Mark. | Modulus and Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 10/22 | 6589 | 6×1⅜ | *.499 | 40.000 | 73.700 | 23.8 | 50.5 | .....60° b'ke | ... | ... | ..... | ½ cup silky edges, pitted center |
| 7/29 | " | ¾ round | ..... | 47.910 | 73.670 | 23.3 | 44.8 | 18″ sq..... | .23 | .77 | ..... | Irregular silky. |
| 7/30 | " | " | ..... | 48.310 | 74.510 | 22.8 | 46.4 | " ..... | ... | ... | ..... | " |
| " | " | " | ..... | 48.630 | 74.510 | 22.4 | 42.5 | " ..... | ... | ... | 29.600.000 | " |
| 10/22 | 2244? | 3×¾ | †.668 | 45.400 | 74.900 | 9.5 | 12.1 | ..... | ... | ... | ..... | Not broken. |
| " | " | 5×1⅛ | *.499 | 39.100 | 74.200 | 26.1 | 52.7 | ... ..... | ... | ... | ..... | ¾ cup silky. |
| " | " | 7×1 7/16 | †.619 | 34.900 | 69.600 | 27.5 | 42.2 | 170° r'd 2′, b'ke | ... | ... | ..... | ½ " |
| " | 6570 | 4×1 9/16 | *.500 | 38.500 | 74.700 | 25.3 | 56.0 | ..... ... | ... | ... | ..... | ⅞ " |
| 7/3 | " | ¾ round | ..... | 46.920 | 71.790 | 23.1 | 46.4 | 18″ sq..... | .22 | .63 | ..... | ½ " |
| " | " | " | ... . | 45.750 | 70.940 | 24.3 | 48.5 | " ..... | ... | ... | 30.260.000 | " |
| 10/22 | 6567 | 4×13/16 | *.450 | 40.200 | 74.900 | 25.8 | 58.9 | ..... | ... | ... | ..... | ¾ " |
| 7/23 | " | ¾ round | ..... | 47.980 | 75.180 | 24.3 | 50.1 | 18″ sq..... | .24 | .91 | ..... | Irregular silky. |
| " | " | " | ..... | 48.200 | 74.950 | 24.5 | 49.1 | ..... | ... | ... | 29.760.000 | |
| 10/22 | 6617 | 3×1¼ | †.621 | 38.500 | 69.600 | 26.9 | 39.4 | ..... .... | ... | ... | ..... | ½ cup silky. |
| 8/8 | " | ¾ round | ..... | 47.530 | 72.900 | 24.4 | 42.2 | 18″ sq..... | .22 | .67 | ..... | Nearly perfect cup, silky. |
| " | " | " | ..... | 48.530 | 72.460 | 25.5 | 43.4 | " .... .. | ... | ... | 29.130 000 | " |
| 10/22 | 6597 | 6×1⅝ | †.552 | 39.500 | 74.300 | 27.3 | 37.5 | 180° r'd 1″ b'ke | ... | ... | ..... | ½ cup silky. |
| 8/1 | " | ¾ round | ..... | 45.710 | 70.330 | 23.2 | 40.2 | 18″ sq.. ..... | .20 | .57 | ..... ... | Irregular silky. |
| " | " | " | . ... | 46 880 | 71.030 | 21.1 | 40.2 | " ..... | ... | ... | 29.530.000 | " |
| 10/22 | ? | 7×2 3/16 | †.626 | 34.700 | 64.900 | 23.1 | 37.1 | Specimen .975×.642 | | | ..... | ⅓ cup, ¼ crystalline on skin edge, remainder pitted. |

In the tests for eyebars (table opposite page 128), the full sized bars were ordered to Athens. From each bar a piece 18″ long was cut, from which the specimen tests were planed out with location in bar as shown by sketch and marks carried out against the tests.

Then the long bars were cut in two and eyebars made from them for test. In case of 3″x$\frac{3}{4}$″ and 4″x$\frac{13}{16}$″ bars, this was done with both pieces of bar and in other cases only one eyebar was made from each size. Hence, it will be seen that the chain is complete, viz., $\frac{3}{4}$″ round tests of ingots at mill—specimen tests of the finished bar from outside and center of bar and finally the test of the completed eyebar made from same bar.

It is to be noted that the small eyebars tested at Fairbanks & Co. are off from same original bar as those of same size sent to Edge Moor, although the excesses in sectional area of head are different.

Eyebars No. 1 to No. 6 inclusive, were submitted in accordance with the specifications, and were tested at Edge Moor Iron Works. Eyebars No. 1a and No. 2a were experimental bars sent to Fairbanks & Co. for test by Union Bridge Co., having less excess section in the eyes.

Eye bar No. 7 was a regular eyebar, which when inspected, showed presence of a cold shut in neck and was rejected.

Experiment was made at blacksmith forge to eliminate the defect, and the appearance of the bar indicated that the work was successful. To test the method however, the bar was reannealed and sent to the Keystone Bridge Company and broken, with results shown in table. The inspector was given special instructions to note appearance of repaired spot during and after test, but no defect was detected.

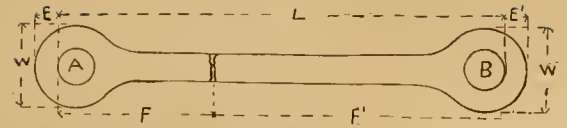
Eyebar No. 8 was a steel lateral bar with upsets and an iron sleeve nut. This bar was not made specially for test.

Eyebars No. 9, No. 10 and No. 11 were selected at random and not made for testing.

The specimen tests given in table from heats 6589, 2244 (?) 6570, 6567, 6617, 6597 and (?) and are on steel, made and used for eyebars of which no full size eye bar tests were obtained.

The following tests show the results of imposing an initial strain of 20,000 lbs. per sq. inch on a car load of eyebars in accordance with specifications. The bars were carefully tested in the Keystone machine. This specification was subsequently waived and no more such tests made.

# TESTS ON FINISHED EYEBARS.

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| Date, 1885. | Test No. | Heat No. | Before Test. Dimensions. Bar. | Length. L. | Pin-hole A. | Pin-hole B. | Sectional Areas. Bar. | Eye A. E. | Eye A. W. | Eye A. % Excess. | Eye B. E'. | Eye B. W'. | Eye B. % Excess. | Elasticity: Limit Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | After Test. Elongations. Hole A. | Hole B. | Bar. Measured Length. | Bar. Amount. | % in Length L. | Reductions. Area of Fracture Sq. Inch. | % of Original Area. | Fracture. Distance from back of Pin-hole F. | Character of Fracture. | Modulus of Elasticity: Pounds. | Remarks on Original Bars. | | Remarks on Tested Bars. | Where Tested. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7 29 | 1 | 6324 | 3 × ¾ | 9′ 3½″ | 3.52″ | 3.52″ | 2.31 | 2.05 | 3.75 | 62.3 | 2.16 | 3.75 | 62.3 | 41,700 | 74,400 | 0.31″ | 0.34″ | No record | 1′-3¼″ | 13.7 | 1.28 | 44.6 | 3′-3½″ | ½ cup, silky edges | .... | Heads planed on sides. | Annealed in open air wood fire. | .... | Edge Moor. |
| " | 2 | 6325 | 4 × 13/16 | 8′-10 5/16″ | 5.77 | 5.77 | 3.33 | 2.96 | 5.07 | 52.3 | 3.06 | 5.08 | 52.5 | 42,300 | 72,100 | 0.51 | 0.38 | " | 0′-11 5/16″ | 11.5 | 1.89 | 43.3 | 1′-11″ | Full cup, silky edges | .... | | | Opened crack ½″ deep on end of Eye A. | " |
| " | 3 | 6339 | 5 × 21/32 | 7′-11 11/16″ | 5.77 | 5.77 | 4.95 | 4.12 | 7.00 | 41.4 | 4.00 | 7.00 | 41.4 | 38,900 | 71,800 | 0.58 | 0.70 | " | 0′-11 5/16″ | 12.1 | 2.89 | 41.6 | 1′-6½″ | ½ finely crystalline, ⅔ silky | .... | | | .... | " |
| " | 4 | 6341 | 5 × 13/16 | 7′-9 7/16″ | 6.27 | 6.27 | 6.00 | 5.05 | 9.20 | 53.3 | 4.96 | 9.21 | 53.5 | 44,400 | 78,500 | 0.69 | 0.61 | " | 0′-11 5/16″ | 12.2 | 3.59 | 10.1 | 2′-4½′ | Silky cup | .... | | | Cold shut in Eye A opened | " |
| " | 5 | 6290 | 6 × 1 | 6′-6⅛″ | 7.27 | 7.27 | 6.00 | 4.76 | 8.77 | 46.1 | 4.76 | 8.70 | 45.0 | 38,300 | 65,400 | ... | 0.54 | " | 0′-3″ | 0.4 | .. | | ..... | Eye end opened, coarsely crystalline | .... | | | Fracture started in initial crack on outside | " |
| " | 6 | 6391 | 7 × 19/8 | 7′-9 3/16″ | 7.27 | 7.27 | 9.77 | 7.32 | 13.50 | 38.2 | 7.24 | 13.41 | 37.2 | 37,900 | 61,300 | 0.58 | 0.65 | " | 0′-3⅜″ | 0.4 | . | | | No fracture | | | | Machine unable to break | " |
| 7 27 | 1B | 6324 | 3 × ¼ | 9′-2 15/16″ | 4.52 | 4.52 | 2.30 | 1.71 | 3.15 | 37.0 | 1.72 | 3.25 | 41.3 | 36,601 | 66,823 | 0.52 | 0.48 | 7′-0″ | 1′-0¾″ | 14.6 | 1.26 | 45.2 | 2′-1½″ | Fine silky | 28,154,000 | | | .... | Fairbanks, N. Y. |
| " | 2B | 6325 | 4 × 12/16 | 8′-15⅝″ | 6.78 | 6.78 | 3.29 | 2.76 | 4.87 | 48.0 | 2.47 | 4.71 | 44.0 | 36,496 | 64,720 | 0.62 | 0.80 | 5′-0″ | 0′-7⅛″ | 11.8 | 1.9 | 42.2 | 1′-9⅜″ | | 30,416,000 | | | .... | " |
| 10/17 1886 | 7 | 6473 | 4 × 15/16 | 34′-9 9/16″ | 1.02 | 5.02 | 3.75 | 3.18 | 5.45 | 45.3 | 3.25 | 5.44 | 45.1 | 35,700 | 63,470 | 0.56 | 0.38 | 20′-0″ | 2′-2⅝″ | 12.4 | 2.69 | 28.3 | 2′-5½″ | Irregular silky edges | 28,236,000 | Heads planed. | Annealed in furnace wood fire. | Heads showed no cold shuts | Keystone. |
| 2 19 | 8 | Long end. 6589 | 3 × 12/16 | 5′-18⅝″ 24′-21⅝″ | 3.02 | 3.02 | 2.44 | 2.48 | 4.78 | 95.9 | 2.48 | 4.70 | 92.6 | 43,200 | 67,700 | 0.25 | 0.19 | 4′-0″ | 0′-7¾″ | 16.1 | 1.27 | 48.0 | Short end 1′-4½″ | Silky cup, showing one separation | .... | Heads not planed. | | Screw threads, 2½″ diam., uninjured, sleeve nut easily unscrewed. | " |
| " | 9 | 6169 | 5 × 1 1/16 | 30′-6¾″ | 5.02 | 4.52 | 5.31 | 4.43 | 7.44 | 40.1 | 4.68 | 7.41 | 40.1 | 41,600 | 66,400 | 0.56 | 0.47 | 20′-0″ | 2′-4″ | 11.7 | 2.58 | 51.5 | 7′-2½″ | Silky cup | .... | | | .... | " |
| " | 10 | 6294 | 5 × 13/16 | 20′-4″ | 5.52 | 5.52 | 5.94 | 4.69 | 9.19 | 54.7 | 4.91 | 10.13 | 70.5 | 40,300 | 64,600 | 0.63 | 0.56 | 16′-0″ | 1′-11″ | 11.9 | 3.07 | 48.3 | 7′-10½″ | Silky center, fine granular edges | .... | | | .... | " |
| " | 11 | 6290 | 5 × 16/16 | 20′-4″ | 5.52 | 5.52 | 6.56 | 5.80 | 9.85 | 50.2 | 5.68 | 9.86 | 50.3 | 40,000 | 70,340 | 0.50 | 0.38 | 16′-0″ | 1′-8¼″ | 10.5 | 5.94 | 9.5 | 1′-4⅛″ | Square across, finely granular throughout | ... | | | .... | " |

## INITIAL TESTS OF EYEBARS WITHIN THE ELASTIC LIMIT.

| Date, 1885. | Mark. | Size of bar. | Length C to C pins. | Size of Pins. A. | Size of Pins. B. | Area of bar. | Pounds strain per sq. in. | Extension in 16′–8″ inches. | Modulus of elasticity. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|
| 10/9 | $L_1$ $M_1$ | 4×1 | 20′–4″ | 3½ | 5¾ | 4.00 | 20,000 | .1355 | 29 520.000 | No permanent set. |
| " | " | " | " | " | " | " | " | .1363 | 29.347.000 | " |
| " | " | " | " | " | " | " | " | .1366 | 29.282.000 | " |
| " | " | " | " | " | " | " | " | .1373 | 29.133.000 | " |
| 10/7 | $L_3$ $M_3$ | 4×⅞ | " | 3½ | 4¾ | 3.50 | " | .1330 | 30.007.000 | " |
| 10/10 | " | " | " | " | " | " | " | .1322 | 30.257.000 | " |
| 10/7 | $L_5$ $M_5$ | 4×⅞ | " | 3½ | 4½ | 3.50 | " | .1312 | 30.490.000 | " |
| 10/10 | " | " | " | " | " | " | " | .1326 | 30.167.000 | " |
| " | " | " | " | " | " | " | " | .1390 | 28.777.000 | " |
| 10/7 | " | " | " | " | " | " | " | .1367 | 29.261.000 | " |
| 10/8 | $L_7$ $M_7$ | 4×27/32 | " | 3½ | 4¼ | 3.37 | " | .1319 | 30.327.000 | " |
| " | " | " | " | " | " | " | " | .1315 | 30.417.000 | |
| 10/10 | " | " | " | " | " | " | " | .1318 | 30.349.000 | Heat No. 6461. Specimen test E –30,750,000. |
| " | " | " | " | " | " | " | " | .1358 | 29.456.000 | " |
| 10/8 | $L_9$ $M_9$ | 4×27/32 | " | 3½ | 4¼ | 3.37 | " | .1357 | 29.478.000 | " |
| 10/10 | " | " | " | " | " | " | " | .1325 | 30.190.000 | " |
| 10/8 | $L_{11}$ $M_{11}$ | 4×27/32 | " | 3½ | 4¼ | 3.37 | " | .1305 | 30.651.000 | " |
| " | " | " | " | " | " | " | " | .1326 | 30.167.000 | " |
| 10/10 | $L_{13}$ $M_{13}$ | 4×⅞ | " | 3½ | 4½ | 3.50 | " | .1327 | 30.143.000 | " |
| " | " | " | " | " | " | " | " | .1350 | 29.260.000 | " |
| 10/7 | " | " | " | " | " | " | " | .1337 | 29.917.000 | " |
| " | " | " | " | " | " | " | " | .1310 | 30.534.000 | " |
| 10/7 | $L_{15}$ $M_{15}$ | 4×⅞ | " | 3½ | 4¾ | 3.50 | " | .1328 | 30.120.000 | " |
| " | " | 4×27/32 | " | " | " | 3.37 | " | .1350 | 29.632.000 | |

## SPECIAL TESTS.

### Tests on Full Size Steel Posts.

The following tests were made on posts built from drawings used for members actually in structure.

The "Tension" post was an exact duplicate, as nearly as can be produced in a shop, of four now in the 260 feet anchorage spans, which were proportioned to take alternate tension and compression.

The "Compression" post is the duplicate of the upper section of double length posts used in the Draw span, but, instead of ending at the splice, the latter was put in, and part of lower section extending from splice to center of middle pin was added and the post made flat ended at that end, the half pin hole being omitted.

In the table, the values of modulus of elasticity were computed after receiving the report from Watertown.

## POST MADE FROM TENSION STEEL—WEIGHT, 2,062 LBS.

PINS IN HORIZONTAL POSITION.—PLATE XX.

Length center to center of pinholes, 276″.07.

Gauged length along center line of web plate, 150″.

Test No. 4,217. Sectional Area, 13.16 square inches.

| Applied Loads. | | In gauged length. | | Deflections at middle″ | | E. | Increment of Comp. in″. | Remarks. |
|---|---|---|---|---|---|---|---|---|
| Total lbs. | Lbs. per Sq.″ | Compression in″. | Set in″. | Horizontal. | Vertical. | | | |
| 15,000 | 1,140 | 0 | ..... | 0 | 0 | | | |
| 20,000 | 1,520 | .0014 | ..... | 0 | 0 | 162,860,000 | .0014 | Initial load. |
| 40,000 | 3,040 | .0095 | ..... | 0 | 0 | 48,040,000 | .0081 | |
| 60,000 | 4,560 | .0180 | ..... | 0 | 0 | 38,000,000 | .0085 | |
| 80,000 | 6,080 | .0263 | ..... | 0 | 0 | 34,677,000 | .0083 | |
| 100,000 | 7,600 | .0345 | .0005 | 0 | 0 | 33,044,000 | .0082 | |
| 110,000 | 8,360 | .0389 | ..... | 0 | 0 | 32,240,000 | .0044 | |
| 120,000 | 9,120 | .0427 | ..... | 0 | 0 | 32,040,000 | .0038 | |
| 130,000 | 9,880 | .0469 | ..... | .01 | 0 | 31,600,000 | .0042 | |
| 140,000 | 10,640 | .0511 | ..... | .01 | 0 | 31,233,000 | .0042 | |
| 150,000 | 11,400 | .0554 | ..... | .02 | 0 | 30,867,000 | .0043 | |
| 160,000 | 12,160 | .0596 | ..... | .02 | 0 | 30,605,000 | .0042 | |
| 170,000 | 12,920 | .0640 | ..... | .02 | .02 | 30,281,000 | .0054 | |
| 180,000 | 13,680 | .0684 | ..... | .02 | .02 | 30,001,000 | .0044 | |
| 190,000 | 14,440 | .0726 | ..... | .03 | .02 | 29,834,000 | .0042 | |
| 200,000 | 15,200 | .0770 | .0016 | .03 | .02 | 29,612,000 | .0044 | |
| | | | Set + | 0 | .02 | ..... | ..... | Permanent movement of eyebar ends, A=.02″, B=.00½″. |
| 210,000 | 15,960 | .0812 | ..... | .04 | .03 | 29,484,000 | .0042 | |
| 220,000 | 16,720 | .0855 | ..... | .04 | .03 | 29,337,000 | .0043 | |
| 230,000 | 17,480 | .0898 | ..... | .05 | .04 | 29,200,000 | .0043 | |
| 240,000 | 18,240 | .0941 | ..... | .05 | .04 | 29,077,000 | .0043 | |
| 250,000 | 19,000 | .0984 | ..... | .05 | .04 | 28,962,000 | .0043 | Movement of eyebar ends under load. A=.02″, B=.00½″. |
| 260,000 | 19,760 | .1025 | ..... | .05 | .04 | 28,929,000 | .0041 | |
| 270,000 | 20,520 | .1068 | ..... | .06 | .04 | 28,822,000 | .0043 | |
| 280,000 | 21,280 | .1109 | ..... | .06 | .04 | 28,783,000 | .0041 | |
| 290,000 | 22,040 | .1153 | ..... | .06 | .04 | 28,671,000 | .0044 | |
| 300,000 | 22,800 | .1195 | .0024 | .06 | .05 | 28,620,000 | .0042 | |
| | | | Set + | .02 | .03 | | | |
| 310,000 | 23,560 | .1235 | ..... | .06 | .05 | 28,618,000 | .0040 | |
| 320,000 | 24,320 | .1279 | ..... | .07 | .05 | 28,522,000 | .0044 | |
| 330,000 | 25,080 | .1323 | ..... | .07 | .05 | 28,436,000 | .0044 | |
| 340,000 | 25,840 | .1365 | ..... | .08 | .06 | 28,398,000 | .0042 | |
| 350,000 | 26,600 | .1412 | ..... | .09 | .06 | 28,258,000 | .0047 | Snapping sound. |
| 360,000 | 27,360 | .1456 | ..... | .09 | .06 | 28,187,000 | .0044 | |
| 370,000 | 28,120 | .1503 | ..... | .10 | .06 | 28,065,000 | .0047 | |
| 380,000 | 28,800 | .1549 | ..... | .11 | .06 | 27,890,000 | .0046 | Under load. A=.03″, B=.00½″. |
| 390,000 | 29,640 | .1600 | ..... | .12 | .06 | 27,787,000 | .0051 | |
| 400,000 | 30,400 | .1646 | .0044 | .13 | .07 | 27,702,000 | .0046 | |
| | | | Set + | .04 | .06 | ..... | ..... | Permanent movement of eyebar ends, A=.02″, B=.00½″. |
| 410,000 | 31,150 | .1702 | ..... | .14 | .09 | 27,453,000 | .0056 | |
| 420,000 | 31,910 | .1748 | ..... | .15 | .09 | 27,383,000 | .0046 | |
| 430,000 | 32,670 | .1749 | ..... | .16 | .10 | 28,019,000 | .0001 | |
| 440,000 | 33,430 | .1854 | ..... | .18 | .10 | 27,048,000 | .0105 | |
| 450,000 | 34,190 | .1920 | ..... | .20 | .10 | 26,712,000 | .0066 | Under load, A=.06″, B=.00½″. |
| 460,000 | 34,950 | .1980 | ..... | .23 | .11 | 26,477,000 | .0060 | |
| 470,000 | 35,710 | .2045 | ..... | .25 | .12 | 26,192,000 | .0065 | |
| 480,000 | 36,470 | .2120 | ..... | .30 | .13 | 25,805,000 | .0075 | |
| 490,000 | 37,230 | .2205 | .0180 | .35 | .16 | 25,326,000 | .0085 | |
| | | | Set + | .15 | .11 | ..... | ..... | Permanent movement of eyebar ends, A=.04″, B=.00½″. |
| 500,000 | 37,990 | ..... | ..... | .43 | .17 | | | |
| 505,000 | 38,370 | ..... | ..... | .50 | .20 | | | |
| 510,000 | 38,750 | ..... | ..... | .57 | .23 | | | |
| 515,000 | 39,130 | ..... | ..... | .65 | .24 | ..... | ..... | Ultimate strength. |
| 513,000 | ..... | ..... | ..... | .70 | .26 | | | |
| 515,000 | ..... | ..... | ..... | .83 | .32 | | | |
| ..... | ..... | ..... | ..... | 1.06 | .33 | | | |
| 500,000 | ..... | ..... | ..... | 1.58 | .37 | | | |
| 440,000 | ..... | ..... | ..... | 2.50 | | | | |

Post failed by buckling the eyebar sections at end A opposite the outside ends of the side plates

## POST MADE FROM COMPRESSION STEEL—WEIGHT, 1,803 LBS

PIN IN VERTICAL POSITION.—PLATE XX.
Length center of pinhole to outside, 286″.57.
Gauged length along center line of upper web plate, 150.″
Test, No. 4,218. Sectional area, 12.95 square inches.

| Applied Loads. | | In gauged length. | | Deflections a middle | | E. | Increment of Comp. | Remarks. |
|---|---|---|---|---|---|---|---|---|
| Total lbs. | Lbs. per sq. ″. | Compression in ″. | Set in ″. | Horizontal. | Vertical. | | | |
| 15,000 | ..... | 0 | 0 | 0 | 0 | ..... | ..... | Initial load. |
| 20,000 | 1.540 | .0019 | ..... | 0 | 0 | 121.579.000 | ..... | |
| 40,000 | 3.090 | .0101 | ..... | 0 | 0 | 45.891.000 | .0082 | |
| 60,000 | 4.630 | .0182 | ..... | 0 | 0 | 38.159.000 | .0081 | |
| 80,000 | 6.180 | .0263 | ..... | 0 | 0 | 35.247.000 | .0081 | |
| 100.000 | 7.720 | .0343 | .0003 | 0 | 0 | 33.761.000 | .0080 | |
| 110,000 | 8.490 | .0378 | ..... | 0 | .02 | 33.691.000 | .0035 | |
| 120,000 | 9.270 | .0419 | ..... | 0 | .02 | 33.186.000 | .0041 | |
| 130,000 | 10.040 | .0458 | ..... | 0 | .03 | 32.882.000 | .0039 | |
| 140.000 | 10.810 | .0500 | ..... | .01 | .03 | 32.4 0.000 | .0042 | |
| 150,000 | 11.580 | .0540 | ..... | .01 | .04 | 32.167.000 | .0040 | |
| 160,000 | 12.360 | .0582 | ..... | .01 | .04 | 31.856.000 | .0042 | |
| 170,000 | 13.130 | .0623 | ..... | .01 | .04 | 31.613.000 | .0041 | |
| 180,000 | 13.900 | .0664 | ..... | .01 | .04 | 31.400.000 | .0041 | Deflection sets. |
| 190,000 | 14.670 | .0705 | ..... | .01 | .05 | 31.213.000 | .0041 | Horizontal Vertical. |
| 200,000 | 15.450 | .0746 | .0001 | .01 | .05 | 31.066.000 | .0041 | 0 0 |
| 210,000 | 16.220 | .0785 | ..... | .01 | .06 | 30.994.000 | .0041 | |
| 220,000 | 16.990 | .0827 | ..... | .01 | .06 | 30.817.000 | .0042 | |
| 230.000 | 17.760 | .0868 | ..... | .01 | .06 | 30.691.000 | .0041 | |
| 240.000 | 18.530 | .0910 | ..... | .01 | .06 | 30.544.000 | .0042 | |
| 250.000 | 19.310 | .0951 | ..... | .01 | .06 | 30.458.000 | .0041 | |
| 260.000 | 20.080 | .0995 | ..... | .01 | .06 | 30.271.000 | .0044 | |
| 270.000 | 20.850 | .1038 | ..... | .01 | .06 | 30.130.000 | .0043 | |
| 280,000 | 21.620 | .1080 | ..... | .01 | .06 | 30.028.000 | .0042 | |
| 290.000 | 22.390 | .1124 | ..... | .01 | .06 | 29.880.000 | .0044 | |
| 300.000 | 23.170 | .1167 | .0001 | .01 | .06 | 29.781.000 | .0043 | 0 .04″. |
| 310.000 | 23.940 | .1212 | ..... | .01 | .06 | 29.629.000 | .0045 | |
| 320.000 | 24.710 | .1266 | ..... | .01 | .06 | 29.277.000 | .0054 | |
| 330.000 | 25.480 | .1297 | ..... | .01 | .06 | 29.468.000 | .0031 | |
| 340.000 | 26.260 | .1345 | ..... | .01 | .06 | 29.286.000 | .0048 | |
| 350.000 | 27.030 | .1390 | ..... | 0 | .04 | 29.169.000 | .0045 | |
| 360.000 | 27.800 | .1434 | ..... | 0 | .04 | 29.079.000 | .0044 | |
| 370.000 | 28.570 | .1479 | ..... | 0 | .03 | 28.976.000 | .0045 | |
| 380.000 | 29.340 | .1529 | ..... | 0 | .03 | 28.783.000 | .0050 | |
| 390.000 | 30.120 | .1575 | ..... | 0 | .03 | 26.686.000 | .0046 | |
| 400.000 | 30.890 | .1622 | .0035 | 0 | .03 | 28.567.000 | .0047 | 0 .03″. |
| 410.000 | 31.660 | .1674 | ..... | 0 | .05 | 28.370.000 | .0052 | |
| 420.000 | 32.430 | .1724 | ..... | 0 | .05 | 28.216.000 | .0050 | |
| 430.000 | 33.210 | .1768 | ..... | 0 | .05 | 28.176.000 | .0044 | |
| 440.000 | 33.980 | .1825 | ..... | 0 | .05 | 27.929.000 | .0057 | |
| 450.000 | 34.750 | .1871 | ..... | 0 | .06 | 27.860.000 | .0046 | |
| 460.000 | 35.520 | .1929 | ..... | 0 | .06 | 27.621.000 | .0058 | |
| 470.000 | 36.290 | .1985 | ..... | 0 | .04 | 27.423.000 | .0056 | |
| 480.000 | 37.070 | .2034 | ..... | 0 | .04 | 27.338.000 | .0049 | |
| 490.000 | 37.810 | .2093 | ..... | 0 | .03 | 27.119.000 | .0059 | |
| 500.000 | 38.610 | .2149 | .0138 | 0 | .08 | 26.950.000 | .0056 | 0 .01″. |
| 510.000 | 39.380 | .2211 | ..... | .01 | .03 | 26.716.000 | .0062 | |
| 520.000 | 40.150 | .2278 | ..... | .01 | .02 | 26.438.000 | .0067 | |
| 530.000 | 40.930 | .2333 | ..... | .01 | .01 | 26.316.000 | .0055 | |
| 540.000 | 41.700 | .2390 | ..... | .01 | .01 | 26.172.000 | .0057 | |
| 550.000 | 42.470 | .2465 | ..... | .01 | .01 | 25.843.000 | .0075 | Snapping sounds. |
| 560.000 | 43.240 | .2540 | ..... | -.02 | 0 | 25.536.000 | .0075 | |
| 570.000 | 44.020 | .2690 | ..... | -.05 | -.01 | 24.546.000 | .0150 | |
| 580.000 | 44.790 | .2787 | ..... | -.10 | -.02 | 24.107.000 | .0097 | |
| 590.000 | 45.560 | .2915 | ..... | -.21 | -.03 | 23.441.000 | .0128 | |
| 597.000 | 46.147 | ..... | ..... | -.40 | 0 | ..... | ..... | Ultimate strength. |
| 560.000 | ..... | ..... | ..... | -.85 | 0 | | | |
| 330.000 | ..... | ..... | ..... | -2.30 | ... | ..... | ..... | Sudden deflection. |

Post failed by deflection in a plane perpendicular to the plane of the axis of the pin.

The deflection increased gradually until the horizontal movement reached -.85″ when the post rapidly deflected to 2.30″; in the meantime the web plates and angles buckled on the concave side at a distance of 8 feet from the pin end.

Pinholes elongated .02″+

Correct, J. E. HOWARD.

F. H. PARKER,
Major Ordinance Dept., U. S. A.,
Commanding.

## SPECIMEN TESTS OF STEEL IN TENSION POST.

| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8". | Reduction of Area, %. | Carbon. | Manganese. | Modulus of Elasticity. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 21 | 6653 | 2½ × 2½ × 5/16 angle | .2943 | 45.300 | 78.320 | 15.8 | 51.2 | .... | .... | 28.811.000 | Specimen 5/16 thick. Fracture irregular. Silky. |
| " | " | 6 × 1½ bar | .4465 | 35.833 | 71.150 | 25.0 | 38.8 | .... | .... | 27.715.000 | Specimen from crop end turned up ¾" round. |
| | " | " | .4465 | 38.475 | 78.160 | 23.3 | 39.8 | .. | .... | 29.643.000 | Specimen from bar, turned up ¾" round. Fracture—silky edges—pitted center. |
| 9 28 | " | ¾ round. Ingot test | .4336 | 49.360 | 72.420 | 23.8 | 44.2 | ... | ... | ..... .. | Very irregular—silky. |
| " | " | " | .4336 | 48.430 | 72.430 | 21.9 | 51.1 | .18 | ? | 29.290.000 | |

## COMPRESSION POST.

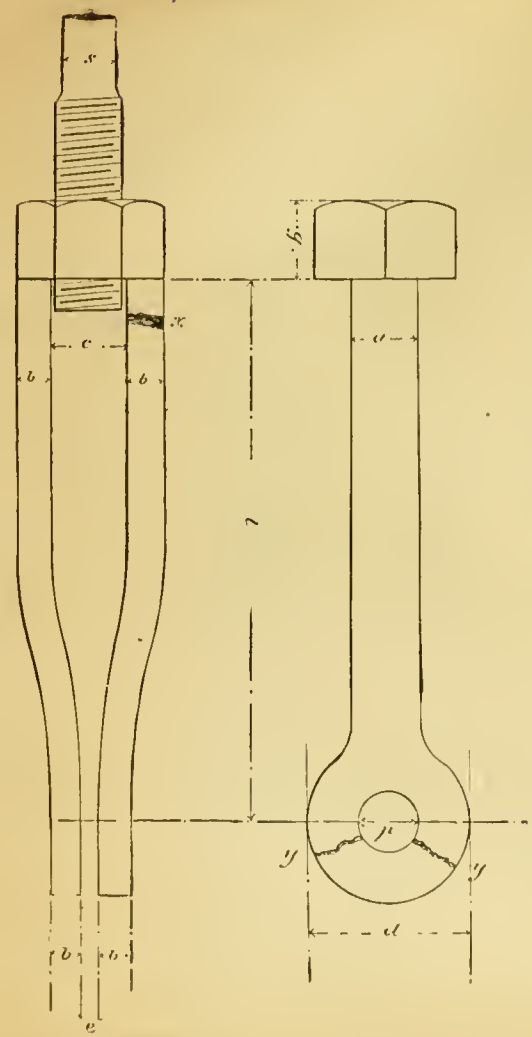
| Date, 1885. | Heat No. | Specimen cut from. | Area of Specimen. | Elastic Limit: Pounds per Sq. Inch. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8". | Reduction of Area, %. | Carbon. | Manganese. | Modulus of Elasticity. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 21 | 6738 | 2½ × 2½ × 5/16 angle | .2895 | 46.290 | 73.920 | 23.3 | 57.1 | .. | .... | 30.285.000 | Fine silky cup. |
| '8 19 | " | ¾ round. Ingot test | .4347 | 51.760 | 83.510 | 18.4 | 30.5 | .34 | .68 | ..... | Silky at 45°—crystal spot. |
| " | " | " | .4394 | 51.210 | 83.730 | 16.6 | 26.8 | .... | .... | 29.840.000 | |
| 1886 1/28 | 4311 | 11 × 5/16 pl. | .611 | 59.900 | 81.600 | 6.0 | 6.2 | ... | ... | ..... .. | Machine could not break. |
| 1885 2 ? | " | ¾ round. Ingot test | ..... | 48.206 | 84.035 | 23.2 | 40.4 | .31 | .78 | ... ..... | Tested at Fairbanks Phila. |
| " | " | " | ..... | ..... | 85.650 | 23.3 | 37.9 | .... | .... | ..... | Tested at Carnegie's Pgh. |
| 2 12 | " | " | ..... | 52.000 | 86.740 | 21.9 | 40.3 | .... | .... | ... ..... | Tested at Steelton. |

The above tests on specimens of steel used in the two posts appear somewhat anomalous, but this may be due to the cases where crop ends only were available for specimens, and to the sometimes imperfect action of the grips of testing machines on very thin specimens. The record is offered, however, as received from inspectors who made all the tests independently of each other.

The following tests were made to ascertain if crucible steel castings had progressed to the point of excellence in small pieces to permit their substitution for difficult and costly forgings.

It was suggested to the writer to use them for clevises, and a set of sample clevises were made by the Pittsburgh Steel Casting Co. The results were not encouraging, as shown in table of tests, but it is to be hoped that in the near future, honeycombing may be eliminated by the use of some device to produce artificial pressure enough to force out the imprisoned gases, and small steel castings be produced as perfect as the many large ones daily turned out.

Iron clevises were finally used, tests having been made on samples of the sizes required in the structure, with results as shown.




## CAST STEEL CLEVISES MADE BY PITTSBURGH STEEL CASTING CO.

Note.—Tests made at Athens on old screw action machine, capacity 150,000 lbs., known to be somewhat inaccurate in readings being out of repair.

| No. of Test. | Size of Rod. | Diam. of Upset. | Dimensions of Clevises, Inches. | | | | | | Original Sectional Areas in Square Inches. | | | | Breaking Strength, per Square Inch, developed. | | | Point of Fracture. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | a | b | d | p | l | e | Rod. | Upset. | Eye. | 2 Wings. | In Rod. | In Clevis Wings. | At Fracture. | | |
| 1 | 1⅜″ sq | 1 13/16 | 2 | ¾ | 5 | 2 | 8 | 1⅝ | 1.89 | 2.58 | 4.5 | 3.0 | 10.600 | 6.720 | 6.720 | At "x" | One clevis broke short off on both wings at "x"—no flaws visible before test. Fracture showed honey-combed structure for about one-half area of cross section. |
| 2 | 1⅜″ sq | 1 13/16 | 2 | ¾ | 5⅜ | 2½ | 6 | 2⅝ | 1.89 | 2.58 | 4.3 | 3.0 | 45.900 | 28.900 | 20.150 | At "yy" | One clevis broke across one eye at "y y"—no flaws visible before test. Sections at fracture honey-combed—one for one-half area—other for one-fourth. |
| 3 | 1⅝″ sq | 2 1/16 | 2½ | ¾ | 5⅞ | 2¼ | 8¼ | 1⅝ | 2.64 | 3.34 | 5.45 | 3.75 | 16.200 | 32.500 | 32.500 | At "x" | Double fracture in one clevis wing at "x"—no flaws visible before test. Sections at fracture honey-combed—one for one-half area—other for one-fifth. |
| 4 | 1⅛″ sq | 1 7/16 | 1¾ | ¾ | 4½ | 2 | .... | ... | 1.27 | 1.62 | 3.75 | 2.63 | 46.900 | 22.660 | 36.660 | Upset | Broke in upset on rod taken from old bridge. Not made at Athens shop. |
| 5 | 1⅛″ sq | 1 7/16 | 1¾ | ¾ | 4½ | 2 | .... | .... | 1.27 | 1.62 | 3.75 | 2.63 | 55.200 | 26.600 | 55.200 | Rod. | |

## WROUGHT IRON CLEVISES MADE BY COFRODE & SAYLOR.

| | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6 | 1⅛″ sq | 1 7/16 | 2 | ¾ | .... | 8 | .... | .... | 1.27 | .... | .... | 3.0 | 51.400 | 21.700 | 51.400 | Rod | Rod had clevises at both ends. |
| 7 | 1⅝″ sq | 2 1/16 | 2⅜ | ¾ | .... | .... | .... | .... | 2.64 | .... | .... | 3.56 | 45.800 | 34.900 | 45.800 | Rod | Rod had clevis at one end and loop at other—loop 1¾×1 3/16 on 2 7/16″ pin. Reduction of area in rod at fracture, 48%. |
| 9 | 1⅜″ sq | 1 13/16 | 2 | ¾ | 5 | 2 | .... | .... | 1.89 | .... | .... | 3. | 44.800 | 28.200 | 41.800 | Rod | Rod had clevis at one end and loop at other—loop 1⅜×1⅛ on 2 7/16″ pin. Reduction of area, 46%. |

## SPECIMEN TESTS OF STEEL PINS ("TENSION" STEEL).

Tests marked Skin were cut as near outside as possible.
Tests marked Center were cut at center.
All specimens turned up round.
9¼ and 7½″ pins forged from 11 inch square and 10 inch square blooms.
All other pins rolled in gothic rolls from 7″×8″ blooms.
Those marked * have ingot test shown elsewhere with tension plates.
Steel made by Penna. Steel Co. and pins made at Carnegie's.

| Date, 1885. | Heat No. | Size of Pin. | Area of Specimen. | Limit of Elasticity per Sq. Inch. | Ultimate Strength per Sq. Inch. | Elongation in 8″%. | Reduction %. | Where cut from. | Carbon. | Manganese. | Remarks on Fracture. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 11/27 | 4732* | 2⅜″ round | .5026 | 39.940 | 68.500 | 25.0 | 44.7 | Skin | ..... | ..... | Fine silky. |
| " | 4778 | 2⅞″ round | .5064 | 41.170 | 71.100 | 24.0 | 46.9 | " | ..... | ..... | Part " |
| 8/10 | " | Ingot | .4336 | 47.770 | 78.220 | 22.8 | 40.1 | ..... | .22 | .58 | |
| " | " | " | .4336 | 49.020 | 78.190 | 22.5 | 39.1 | | | | |
| 11/27 | 4776* | 3⅜″ round | .5026 | 38.655 | 69.970 | 17.6 | 48.7 | Skin | ..... | ..... | Fine silky. |
| " | 4780* | 5⅝″ round | .5052 | 38.756 | 69.730 | 24.5 | 45.3 | Skin | Same | Pin | Irregular—part silky. |
| " | " | " | .5052 | 41.680 | 65.710 | 20.0 | 30.0 | Center | | | Square with granular center. |
| " | 4868* | " | .5115 | 38.840 | 69.795 | 23.0 | 46.4 | Skin | Same | Pin | Shallow—part silky. |
| " | " | " | .5140 | 41.090 | 65.480 | 18.8 | 30.8 | Center | | | Jagged—part silky—center granular. |
| " | " | 4⅞″ round | .5166 | 38.595 | 70.130 | 23.1 | 52.3 | Skin | ..... | ..... | Fine silky. |
| 11/28 | 4776* | 6⅜″ round | .5001 | 39.730 | 69.880 | 22.0 | 52.5 | Skin | Same | Pin | " " |
| " | " | " | .5039 | 41.095 | 66.650 | 20.0 | 24.7 | Center | | | Square—granular throughout. |
| " | 4772* | 6⅝″ round | .5166 | 38.760 | 70.480 | 26.3 | 50.6 | Skin | Same | Pin | Fine silky. |
| " | " | " | .5140 | 42.850 | 67.490 | 18.0 | 29.3 | Center | | | Irregular — center granular. |
| 11/30 | 4649* | 6⅞″ round | .5089 | 39.790 | 69.830 | 26.9 | 53.7 | Skin | Same | Pin | Shallow—part silky. |
| " | " | " | .5039 | 41.035 | 67.490 | 16.3 | 24.7 | Center | | | Irregular — center granular. |
| " | 4644 | 6⅜″ round | .5178 | 38.760 | 72.540 | 25.0 | 45.4 | Skin | Same | Pin | Fine silky. |
| " | " | " | .5115 | 41.080 | 67.490 | 19.5 | 43.4 | Center | | | Shallow — center pitted and granular. |
| 6/5 | " | Ingot | .4525 | 48 190 | 74.920 | 23.5 | 40.5 | ..... | .20 | .76 | Fine silky. |
| " | " | " | .4525 | 49.500 | 75.280 | 25.5 | 40.5 | ..... | ..... | ..... | At 45° silky. |
| 11/28 | 4774* | 7½″ round | .4976 | 41.920 | 72.370 | 30.3 | 53.6 | Skin | Same | Pin | Fine silky. |
| " | " | " | .5064 | 39.040 | 67.510 | 18.1 | 30.4 | Center | | | Edges silky—center granular. |
| " | 4740 | 9¼″ round | .5039 | 40.840 | 71.980 | 26.8 | 54.5 | Skin | Same | Pin | Fine silky. |
| " | " | " | .5026 | 38.860 | 65.470 | 17.5 | 29.8 | Center | | | Edges silky—center coarsely granular. |
| 8/6 | " | Ingot | .4371 | 47.200 | 70.320 | 24.5 | 46.0 | ..... | .20 | .58 | |
| " | " | " | .4371 | 47.540 | 70.230 | 24.3 | 44.1 | | | | |

## SHEARING STEEL PLATES.

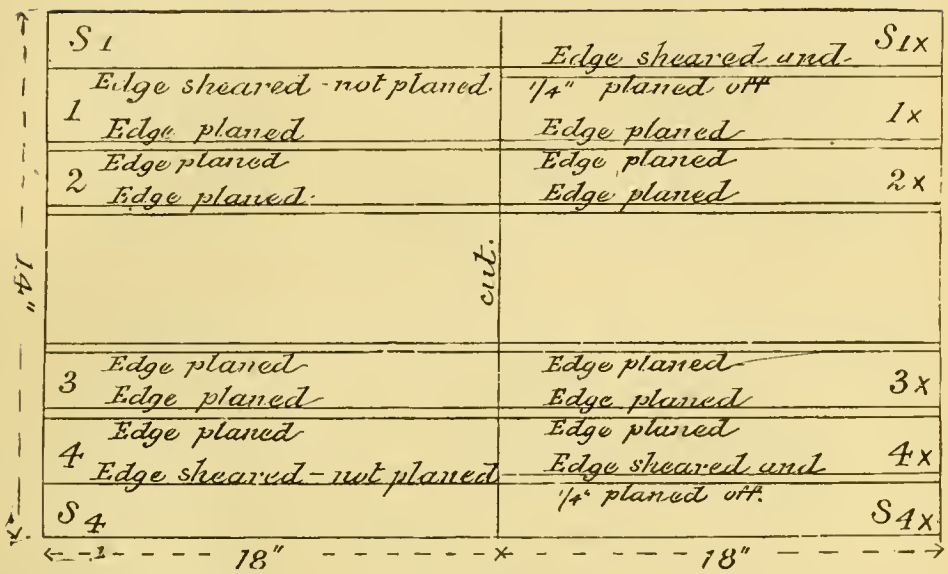
The following tests were made to determine, if possible, the damage done to steel plates by shearing the edges, and to ascertain whether planing off ¼″ from the sheared edges removes the injury to the plate.

The specimens were prepared as indicated in sketch and table, straightened without hammering and broken in 50,000 lb. Olsen lever machine at Athens.

"It was not possible to note the depth to which the shearing affected the material, by inspection of the edges before breaking, nor by any difference in the fracture after pulling; the fracture was in every case but one, entirely silky. One specimen mainly crystalline, was silky on the sheared side as noted in the table."

Steel made at Penna. Steel Co.'s mills and rolled at Elmira Rolling Mill. Slabs 14¼″x1¼″ and 1½″. Steel same grade as used in structure, made under same specification.

Sketch.




| Mark. | Treatment of Specimen. | Specimen cut from. | Area of Specimen. | Limit of Elasticity per sq." lbs. | Ultimate Strength: Pounds per Sq. Inch. | Elongation, % in 8 Inches. | Reduction of Area, %. | Carbon. | Manganese. | Remarks on Fracture. |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 edge sheared, other planed. | Specimens, 14"×5/16" Tension plate. | .3843 | 42.270 | 68.030 | .... | .... | .... | .... | Not completely broken. |
| 2 | Both edges planed. | | .3936 | 41.410 | 66.060 | 24.4 | 49.8 | .... | .... | Silky. Broke in lamination. |
| 3 | Both edges planed. | | .3924 | 41.890 | 67.670 | 25.7 | 51.4 | .... | .... | Silky. Sheared at 45°. Broke 4" from grip. Broke 2" from end. |
| 4 | 1 edge sheared, other planed. | | .3862 | 43.240 | 67.580 | 16.0 | 29.4 | .... | .... | Silky. Broke on sheared edge, first starting to tear in many places. |
| 1x | 1 edge sheared. then planed 1/4", other edge planed. | | .3860 | 41.210 | 68.000 | 24.0 | 49.6 | .... | .... | Silky at 45°. |
| 2x | Both edges planed. | ..... | .3948 | 41.600 | 68.000 | 23.2 | 49.8 | .... | .... | A little crooked—laminated. |
| 3x | Both edges planed. | ..... | .3924 | 41.910 | 68.430 | 23.5 | 49.1 | .... | .... | " " " silky irregular. |
| 4x | Same as 1x. | ..... | .3875 | 41.290 | 68.130 | 23.5 | 53.1 | .... | .... | " " " " cup-fine. |
| 4561 | Ingot test. | 3/4" round | ..... | 51.670 | 74.570 | 25.9 | 55.6 | .22 | .65 | Silky—part crystal. Break started on sheared edge, broke with more snap than 5/16" pieces. |
| 1 | Treatment same as in other specimens of same marks. | Specimens, 14"×7/16" Tension plate. | .4383 | 47.450 | 74.370 | 19.1 | 36.2 | .... | .... | Silky—concave—10% crystal, broke 3" from grip. |
| 2 | | | .4348 | 43.240 | 72.920 | 23.1 | 52.1 | .... | .... | Ditto. |
| 3 | | | .4328 | 42.060 | 72.650 | 21.6 | 49.1 | .... | .... | Irregular silky—part fine crystal—break started in many points on sheared edge, broke 2" from grip. |
| 4 | | | .4310 | 41.070 | 74.230 | 16.2 | 31.0 | .... | .... | Silky cup—part very fine crystal. |
| 1x | | | .4288 | 43.610 | 74.630 | 23.1 | 50.3 | .... | .... | Irregular cup, dull center, broke 3" from grip. |
| 2x | ..... | | .4308 | 44.330 | 73.800 | 23.1 | 46.8 | .... | .... | |
| 3x | ..... | | .4308 | 44.800 | 75.200 | 20.5 | 50.2 | .... | .... | Ditto. |
| 4x | ..... | | .4308 | 43.640 | 75.430 | 22.2 | 51.9 | .... | ... | Irregular cup. fine silky, broke 3" from end—small lamination in center. |
| $S_1$ | 1 edge sheared, then planed 1/8", other edge planed. | ..... .. | .4350 | 45.370 | 75.270 | 23.0 | 39.6 | .... | .... | Very irregular—fine silky. |
| $S_4$ | Ditto. | ..... | .4300 | 46.290 | 75.800 | 19.6 | 51.2 | .... | .... | Irregular—part fine crystal. |
| 4976 | Ingot test | 3/4" round | ..... | 50.770 | 76.650 | 22.5 | 52.0 | .24 | .77 | |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Treatment same as in other specimens with same marks. | 1″×⅝″ Tension plate. Specimens. ¾″×⅝″. | .4697 | 53.000 (?) | 74.720 | 14.1 | 28.5 | .... | ... | At 45° silky—broke from sheared edge—bent in testing. |
| 2 | | | .4637 | 39.900 | 73.320 | 25.6 | 57.3 | .... | ... | Half cup—silky—regular. |
| 3 | | | .4693 | 41.330 | 73.550 | 16.5 | 50.3 | .... | .... | " " " |
| 4 | | | .4812 | 47.290 | 75.330 | 12.5 | 21.9 | .... | .... | Bent in testing—broke 3″ from end with oblique sheared. Silky. |
| 1x | | | .4656 | 41.230 | 72.600 | 22.7 | 44.6 | .... | .... | Irregular cup—silky—trace very fine crystal. |
| 2x | | | .4681 | 41.600 | 70.900 | 26.0 | 55.6 | .... | .... | Cup silky. |
| 3x | | | .4696 | 42.500 | 71.700 | 20.8 | 50.7 | .... | .... | Irregular silky. |
| 4x | | | .4666 | 42.540 | 73.190 | 20.4 | 54.1 | .... | ... | " " |
| 4796 | Ingot test. | ¾ round | ..... | 50.770 | 76.650 | 22.5 | 52.0 | .24 | .77 | |
| 1 | Treatment same as in other specimens with same marks. | 1″×5/16″ Compression plate. Specimens. 1¼″×5/16″. | .3968 | 48.760 | 75.100 | 15.5 | 23.8 | .... | .... | Very rough sheared. Silky, irregular bent in testing. Break started on sheared edge. |
| 2 | | | .3968 | 47.370 | 75.100 | 23.4 | 42.9 | .... | .... | At 45° silky. |
| 3 | | | .3906 | 48.130 | 74.900 | 25.9 | 44.2 | .... | .... | A little crooked—irregular, very silky. |
| 4 | | | .3869 | 49.740 | 75.220 | 13.8 | 20.4 | .... | .... | Very rough sheared. Broke suddenly at 45°—silky. Started breaking at sheared edge. |
| 1x | | | .3906 | 50.700 | 75.660 | 22.5 | 44.6 | .... | .... | Started on planed edge at 45° silky. |
| 2x | | | .3918 | 46.960 | 74.780 | 23.0 | 46.1 | .... | .... | Piece a little crooked. Part cup silky. |
| 3x | | | .3869 | 48.070 | 75.230 | 22.9 | 44.8 | .... | .... | Piece quite crooked. At 45° silky. |
| 4x | | | .3869 | 48.900 | 74.630 | 20.8 | 40.5 | .... | .... | Piece a little crooked. Very irregular silky. |
| 4340 | Ingot test. | | ..... | 52.340 | 81.260 | 22.3 | 41.3 | .27 | .85 | |
| 1 | Treatment same as in other specimens with same marks. | 1″×7/16″ Compression plate. Specimens. 1″×7/16″. | .4466 | 49.490 | 72.820 | 11.0 | 19.4 | .... | .... | Sheared very rough—intermediate seam—crooked—started in dent on sheared edge. Silky at 45°. |
| 2 | | | .4365 | 48.570 | 74.470 | 21.9 | 45.1 | .... | .... | Silky at 45°. |
| 3 | | | .4345 | 48.790 | 73.880 | 23.3 | 50.0 | .... | .... | " " " |
| 4 | | | .4383 | 51.220 | 74.600 | 14.2 | 23.9 | .... | .... | Broke in hollow spot on sheared edge—oblique fracture silky. |
| 1x | | | .4333 | 48.250 | 71.160 | 25.2 | 44.9 | .... | .... | Irregular silky—broke in punch mark. |
| 2x | | | .4354 | 47.320 | 73.830 | 28.5 | 44.9 | .... | ... | Part cup—fine silky. |
| 3x | | | .4369 | 48.060 | 74.150 | 21.6 | 42.9 | .... | .... | Broke 1″ from grip. irregular silky. |
| 4x | | | .4358 | 47.060 | 74.580 | 24.6 | 48.8 | .... | .... | At 45° silky. |
| $S_2$ | One edge sheared and then planed ⅛″—other edge planed. | | .4330 | 52.510 | 78.080 | 20.5 | 49.7 | .... | .... | Irregular cup silky. Began tearing on sheared edge. |
| $S_4$ x | Ditto. | | .4330 | 52.050 | 78.540 | 23.5 | 47.8 | .... | .... | Irregular cup silky. bent in tearing. |
| 4340 | Ingot test. | ¾ round | ..... | 52.340 | 81.260 | 22.3 | 41.3 | .27 | .85 | |
| 1 | Treatment same as in other specimens with same marks. | 1″×⅝″ Compression plate. Specimens. ¾″×⅝″. | .4781 | 60.780 (?) | 74.200 | 21.2 | 18.1 | .... | .... | Rough piece with seams ¾ crystalline towards planed side. |
| 2 | | | .4636 | 39.710 | 74.120 | 24.2 | 48.6 | .... | .... | Partial cup—part crystalline |
| 3 | | | .4649 | 41.300 | 74.320 | 24.3 | 49.6 | .... | .... | Flat cup silky. |
| 4 | | | .4634 | 51.790 | 74.450 | 20.6 | 11.6 | | | Sheared rough with seams broke in seam. Elastic Limit taken with dividers. |
| 1x | | | .4584 | 40.790 | 74.160 | 21.2 | 50.2 | .... | .... | Silky—regular. |
| 2x | | | .4607 | 41.040 | 73.150 | 23.5 | 36.0 | .... | .... | " irregular—traces of crystal. |
| 3x | | | .4642 | 41.170 | 73.600 | 25.5 | 52.6 | .... | .... | Cup silky. |
| 4x | | | .4601 | 41.080 | 74.450 | 23.7 | 44.4 | .... | .... | Half cup silky—broke in punch mark |
| 340 | Ingot test. | ¾ round | ..... | 52.340 | 81.260 | 22.3 | 41.3 | .27 | .85 | |

## HAMMERING STEEL PLATES.

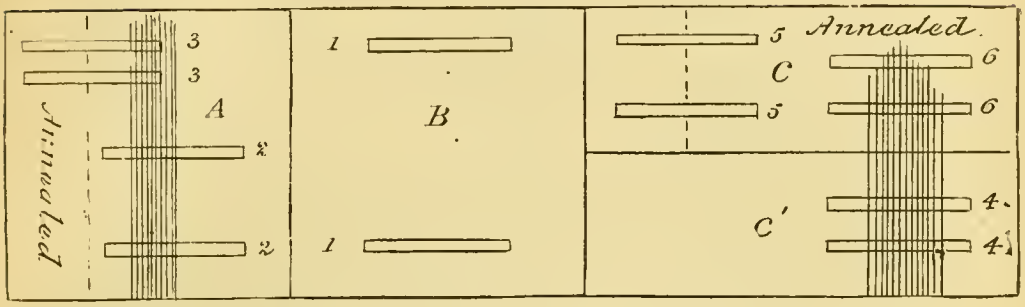



Plate cut up into parts A, B, C and C'.

A. Hammered while hot. (Heated between dull red plates.) After hammering and cooling, part up to dotted line was reheated and cooled in ashes between two hot plates.

B. Normal plate, not hammered.

C. Hammered while cold. Part up to dotted line afterward annealed as in case of A.

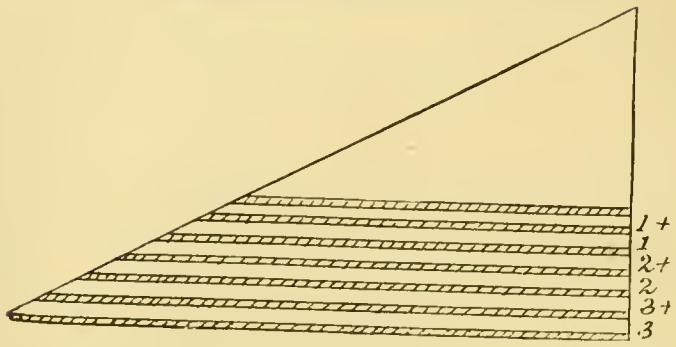
C'. Hammered cold. Not annealed.

Plate originally buckled as indicated in shaded parts. Hammered only enough to straighten out.

| 1885 | Test. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit per Sq. Inch: Pounds. | Ultimate Strength per Sq. Inch: lbs. | Elongation in 8", %. | Reduction of Area, %. | | Remarks on Fracture. |
|---|---|---|---|---|---|---|---|---|---|
| 9/18 | No. 1 | 24×⅜ | .358 | 52.200 | 85.900 | 22.1 | 39.9 | | Slightly laminated. |
| " | " 1 | " | " | 56.700 | 86.300 | 22.9 | 47.7 | | " |
| " | " 2 | " | .358 | 55.800 | 90.100 | 17.3 | 46.1 | | Part cup silky. |
| " | " 2 | " | .357 | 55.200 | 90.300 | 17.5 | 36.4 | | " " " |
| " | " 3 | " | .357 | 56.000 | 89.600 | 18.6 | 37.6 | Broke near edge of annealed portion. | " " " |
| " | " 3 | " | " | 56.000 | 88.200 | 18.1 | 36.7 | " " | " " " |
| " | " 4 | " | .360 | 51.900 | 87.000 | 21.9 | 40.8 | | Slightly laminated. |
| " | " 4 | " | .361 | 50.900 | 86.100 | 19.8 | 34.3 | | " " |
| " | " 5 | " | .359 | 52.900 | 86.300 | 16.2 | 44.0 | | Part cup silky. |
| " | " 5 | " | .363 | 55.100 | 88.100? | 16.2 | 38.3 | | " " |
| " | " 6 | " | .364 | 53.600 | 85.000 | 20.6 | 35.4 | | Slightly laminated. |
| " | " 6 | " | .363 | 53.700 | 85.400 | 21.6 | 47.6 | | " " |

Tested at Athens.

## HAMMERING STEEL PLATES.




Test pieces Nos. 3 and 3+ taken off when plate was in normal condition. Plate then heated to cherry red and bent ½″ out of plane. Hammered plate till it was entirely covered with hammer marks. Test pieces Nos. 2 and 2+ then cut off. Plate then annealed by placing between two other plates, 1¼″ thick, heated to dull red. All covered with ashes and cinder and left over night to cool. Test pieces Nos. 1 and 1+ then cut off.

| 1885. | Mark. | Specimen cut from. | Area of Specimen: Sq. Inches. | Elastic Limit per Sq. Inch: Pounds. | Ultimate Strength per Sq. Inch: Pounds. | Elongation in 8″. %. | Reduction of Area, %. | Remarks on Fracture. |
|---|---|---|---|---|---|---|---|---|
| 9/26 | No. 3 | 30×⅝ | .457 | 45.700 | 71.500 | 18.1 | 42.0 | Broke at surface. Slag pit section measurement necessarily inaccurate. |
| " | " 3+ | " | .433 | 48.000 | 72.100 | 18.1 | 38.0 | Badly pitted on one side. Calipered as near an average as possible. |
| " | " 2 | " | .389 | 37.600 | 70.700 | 22.5 | 53.7 | Pitted surface planed of. |
| " | " 2+ | " | .389 | 37.500 | ..... | 22.1 | 47.8 | " |
| " | " 1 | " | .398 | 36.100 | 66.800 | 20.3 | 49.8 | " |
| " | " 1+ | " | .389 | 35.500 | 65.900 | 19.6 | 46.0 | " |

Tested at Athens.

The foregoing tests were made at Athens and were occasioned by an accident. A long plate 26″x⅝″ had come in from mill with several buckled spots in it, and the men in the yard took it to the platen and hammered them out without the fact being noticed.

On observing the hammer marks on the plate afterwards, when being bolted up in its proper member, the writer had the member laid aside, hammering cold being distinctly specified against.

The urgent need of completion of the member in question, and impossibility of replacing the plate under three to four weeks, and thus delaying the erection of the bridge that long, made it a serious affair if the plate must be rejected.

To decide the matter, the tests recorded above were made and as a result the plate allowed to go into the member, which was a chord section strained in compression.

The writer desires at this time to acknowledge, with due appreciation, the hearty co-operation of the Union Bridge Co., in his endeavors to make such tests as were necessary to arrive at true conclusions in all cases of doubt, and trusts that the results have proved as satisfactory to them as to himself.

Also to record his satisfaction at the care and painstaking efforts of the several inspecting engineers engaged with him in the work to obtain proper material and workmanship, and in carrying on the special tests.

PLATE XIII.
TRANS, AM. SOC. CIV. ENG'RS.
VOL. XVII, NO. 365.
MOULTON ON
KENTUCKY AND INDIANA BRIDGE.

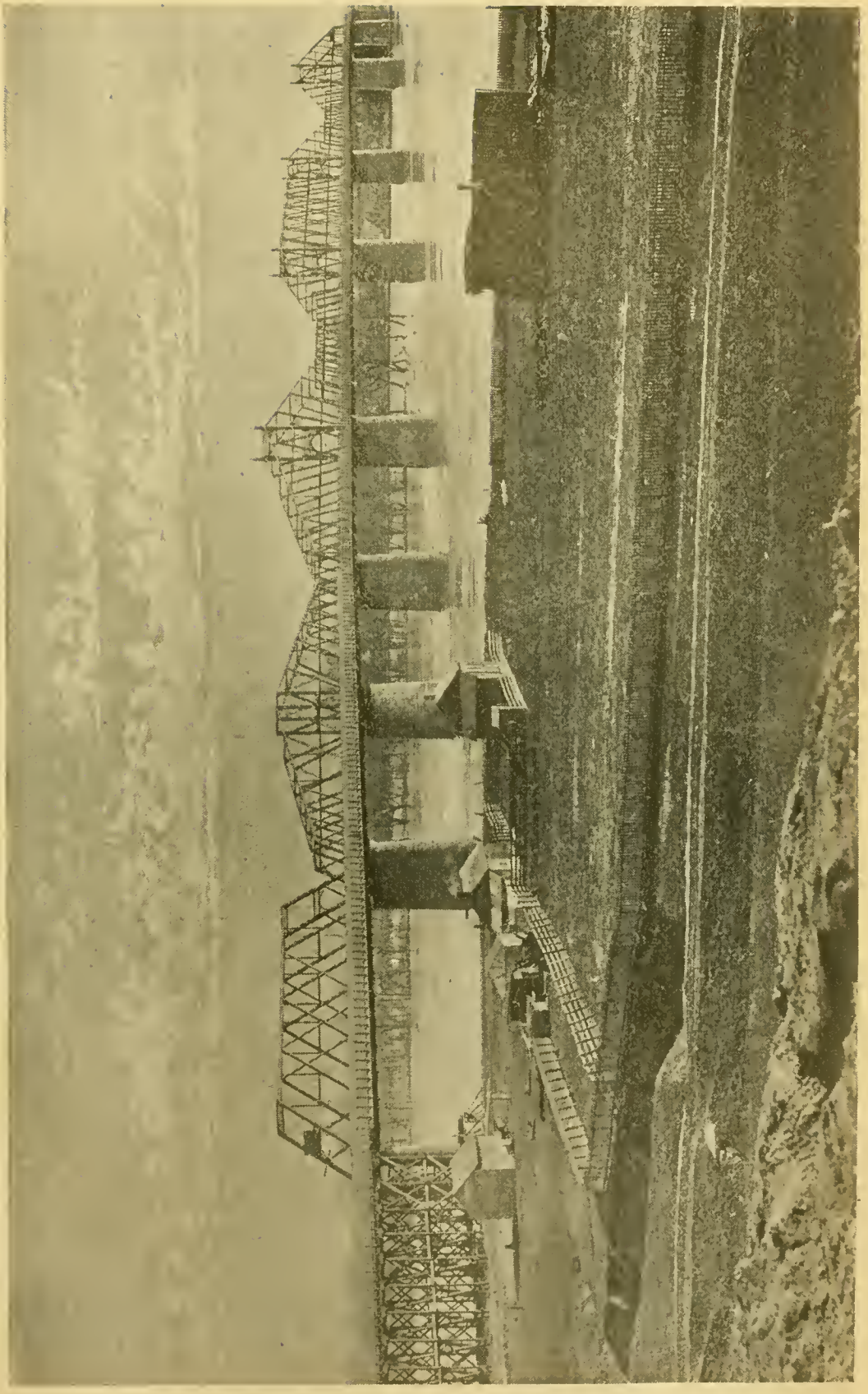


PLATE XIV
TRANS. AM. SOC. CIV ENG'RS
VOL. XVII Nº 365.
MOULTON ON THE
KENTUCKY & INDIANA BRIDGE.

PLATE XV
TRANS AM SOC.CIV ENGRS
VOL XVII Nº 365
MOULTON ON THE
KENTUCKY & INDIANA BRIDGE

*Hydrograph showing Rise and Fall of Ohio River at Bridge Site from Jan. 1st 1875 to July 1st 1886.*

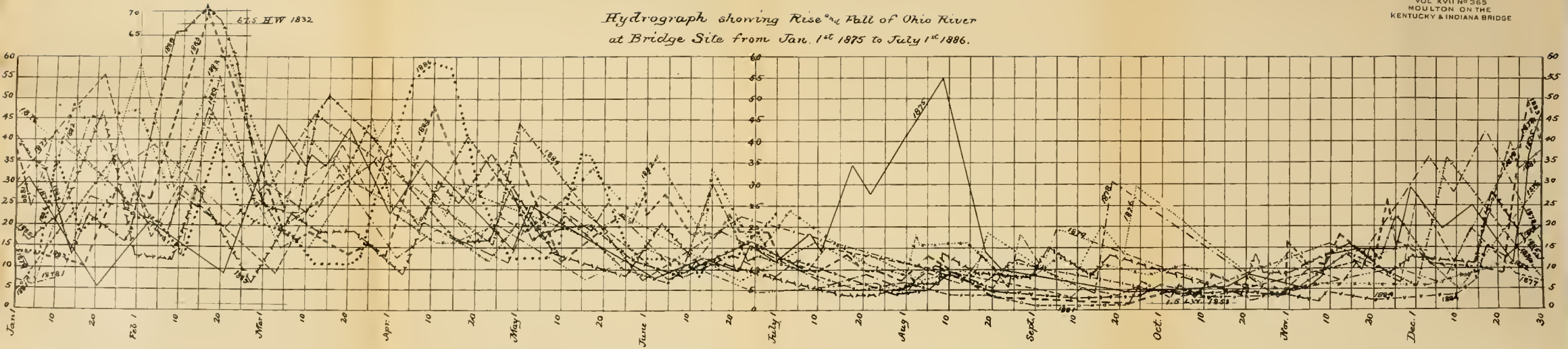
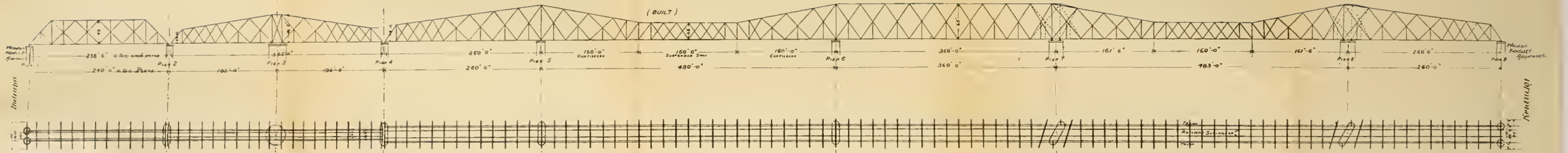


PLATE XVI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVII No. 365
MOULTON ON THE
KENTUCKY & INDIANA BRIDGE

# Kentucky and Indiana Bridge.

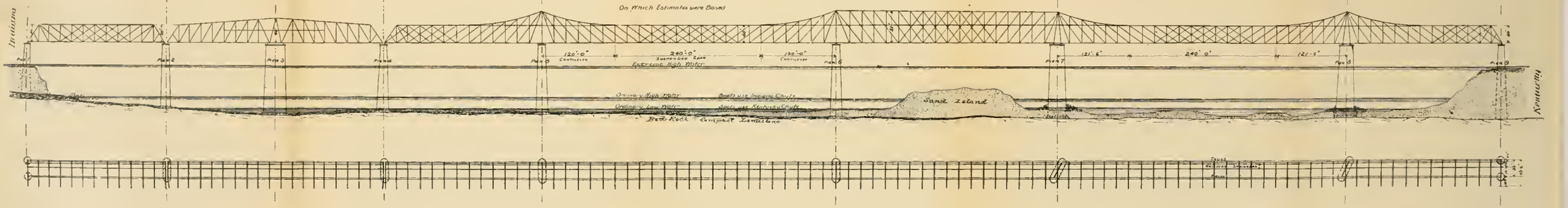
Final Design of Messrs. C. Macdonald and E. Hembeale

(Built)



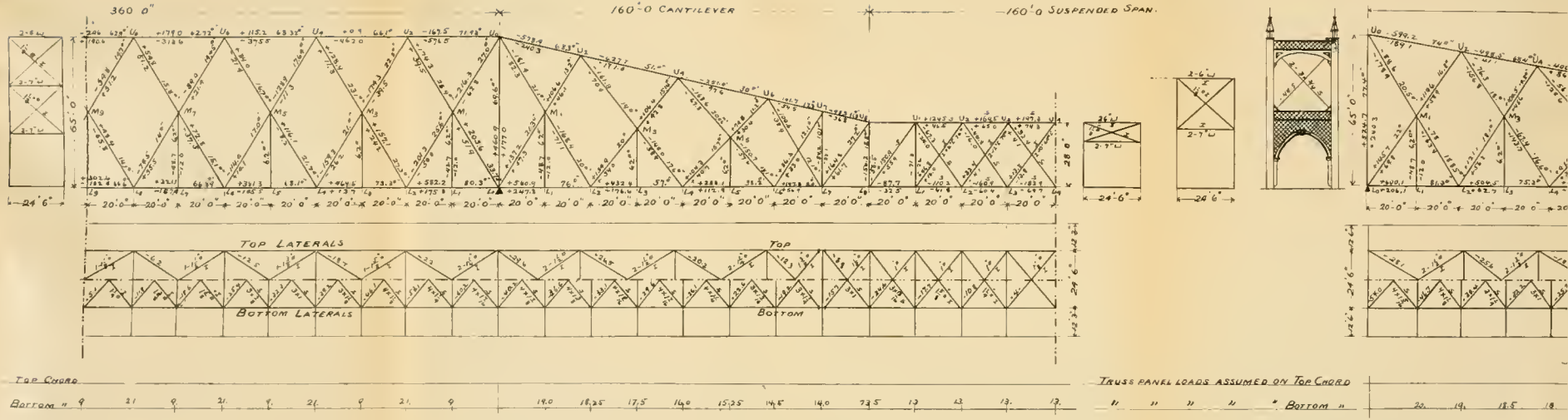

Original Design, C. Shaler Smith, C.E.

On Which Estimates were Based



# Stress Diagram

All loads and resulting stresses given in tons of 2000 lbs. All mater




Live Load for all spans. on R
on R
with

# Kentucky and Indiana Bridge

*...eriid steel unless otherwise marked. In spans where iron is used, iron marked I, and steel S.*

PLATE XVIII
TRANS AM SOC CIV ENGRS
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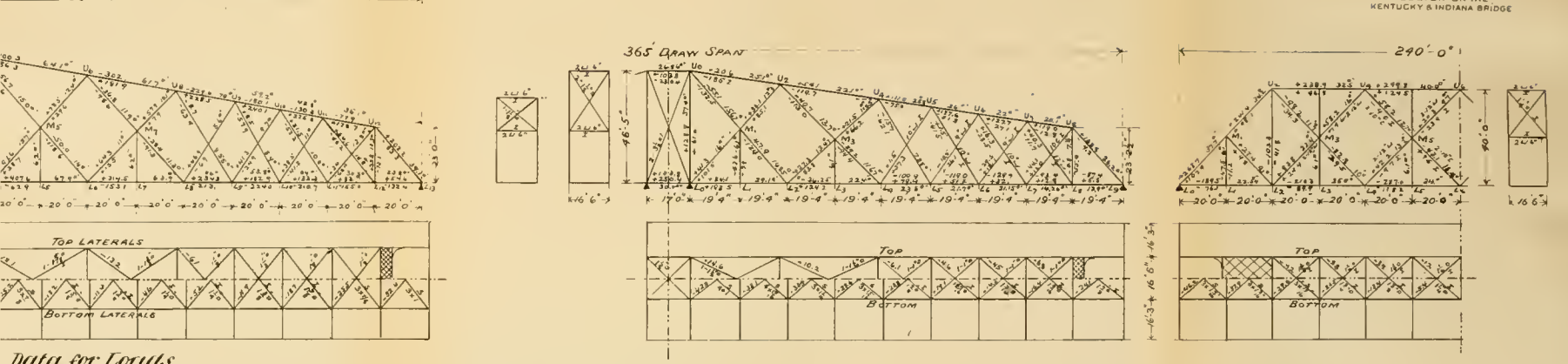
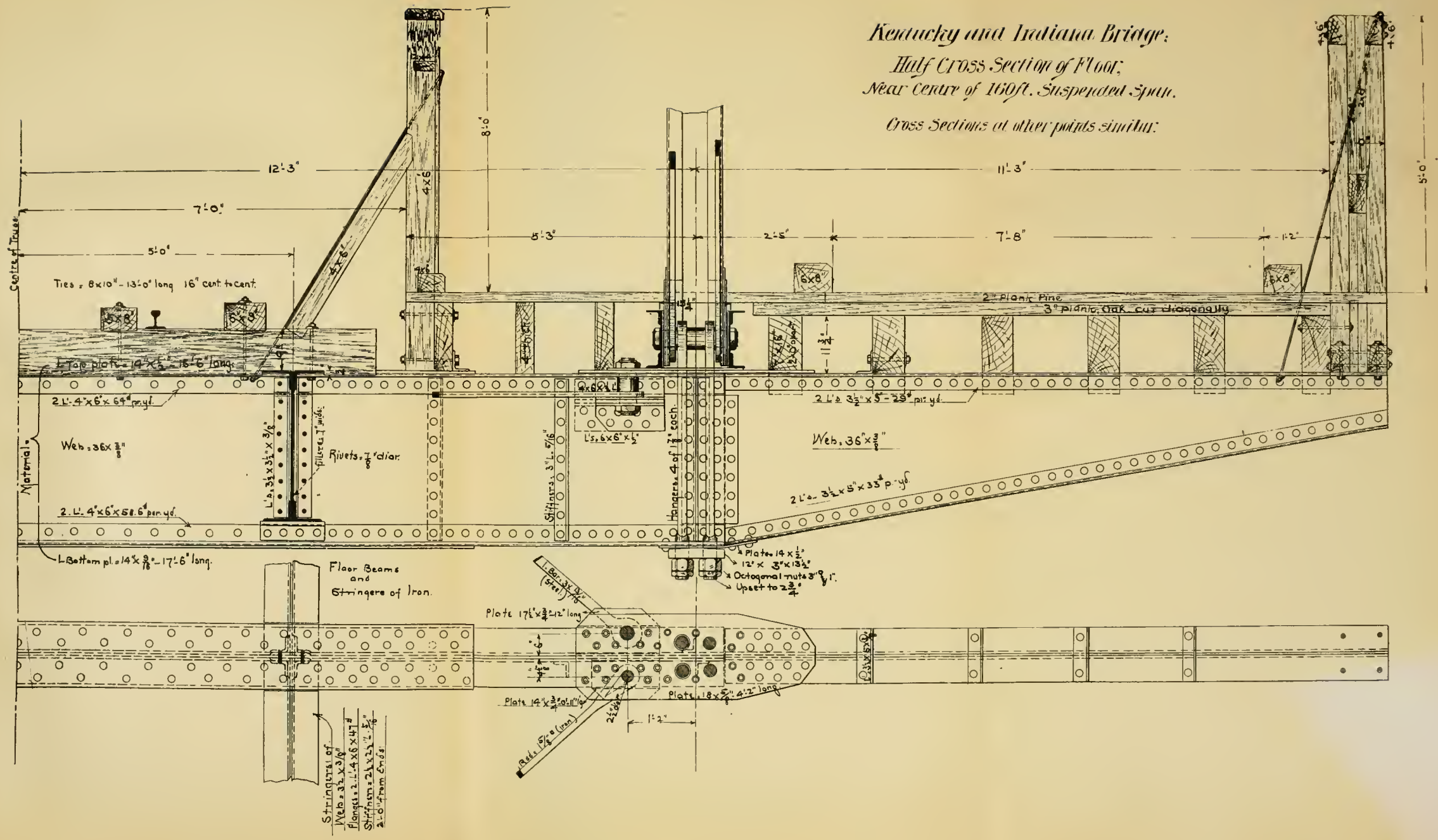
MOULTON ON THE
KENTUCKY & INDIANA BRIDGE



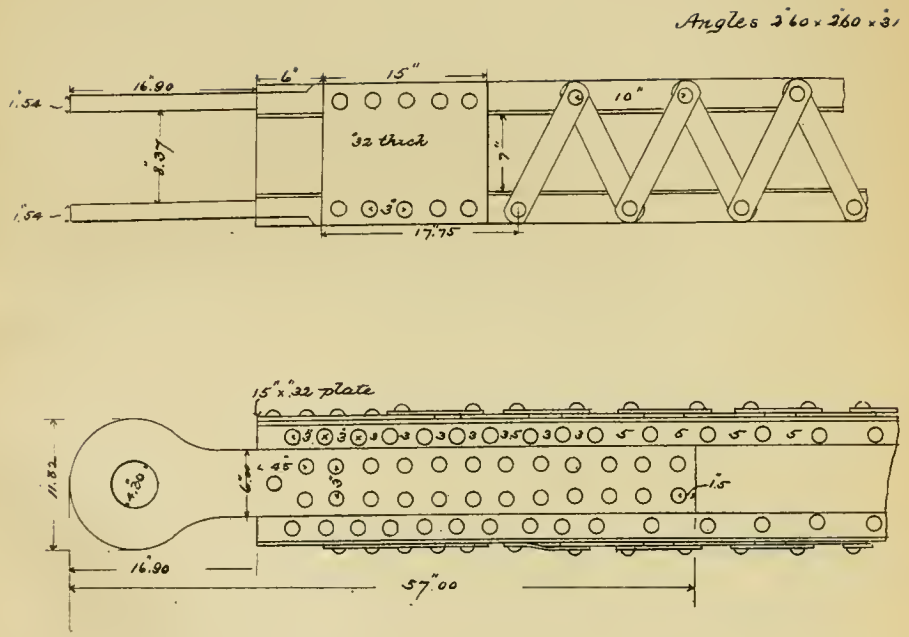

## Data for Loads

| | | |

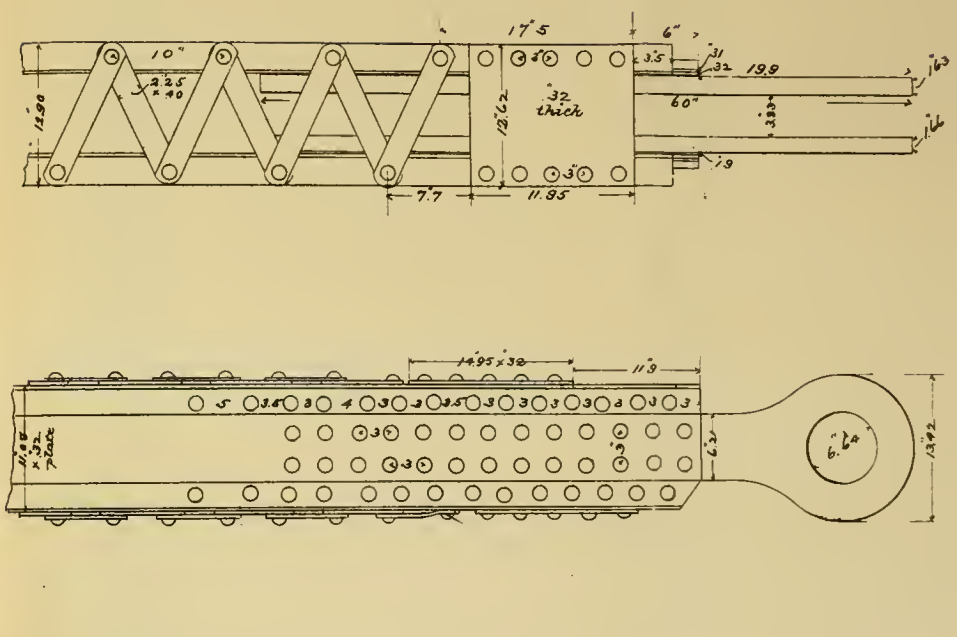
PLATE XIX
TRANS. AM. SOC. CIV. ENG'RS
VOL. XVII No 365.
MOULTON ON THE
KENTUCKY & INDIANA BRIDGE.



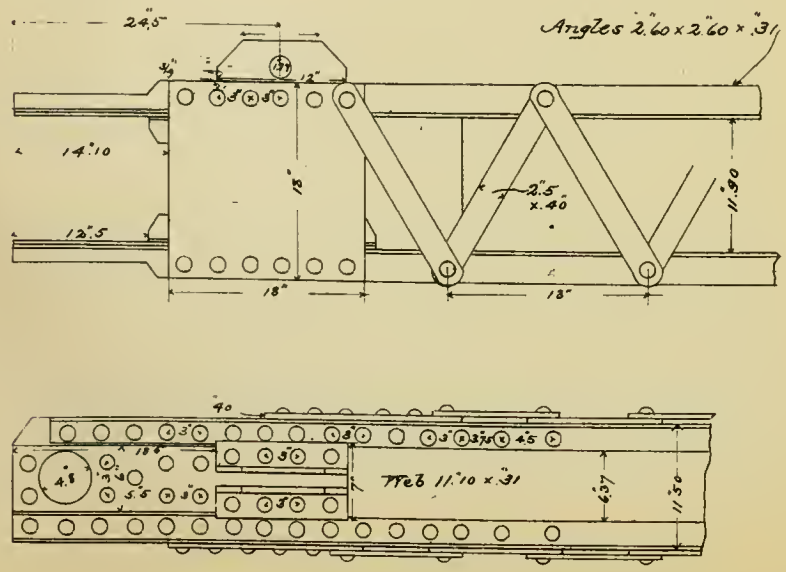
No. 4217 End A.




No. 4217 End B.




No. 4218 End A.




No. 4218 End B.

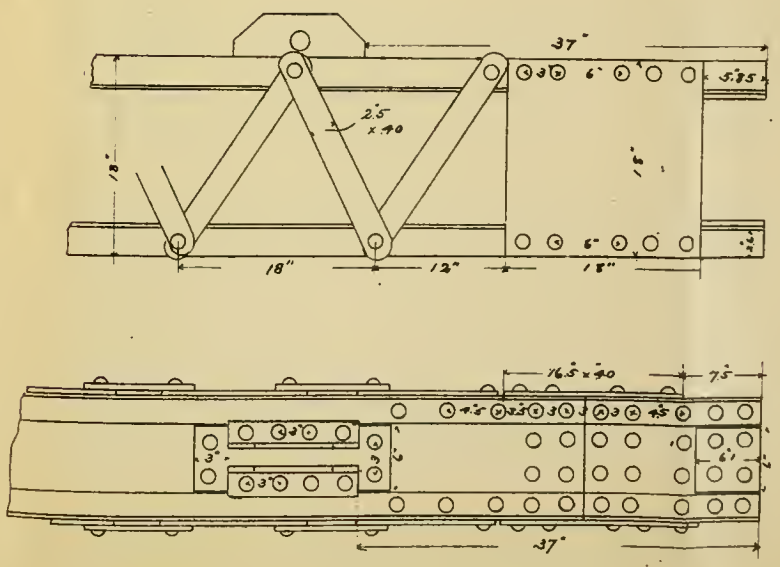
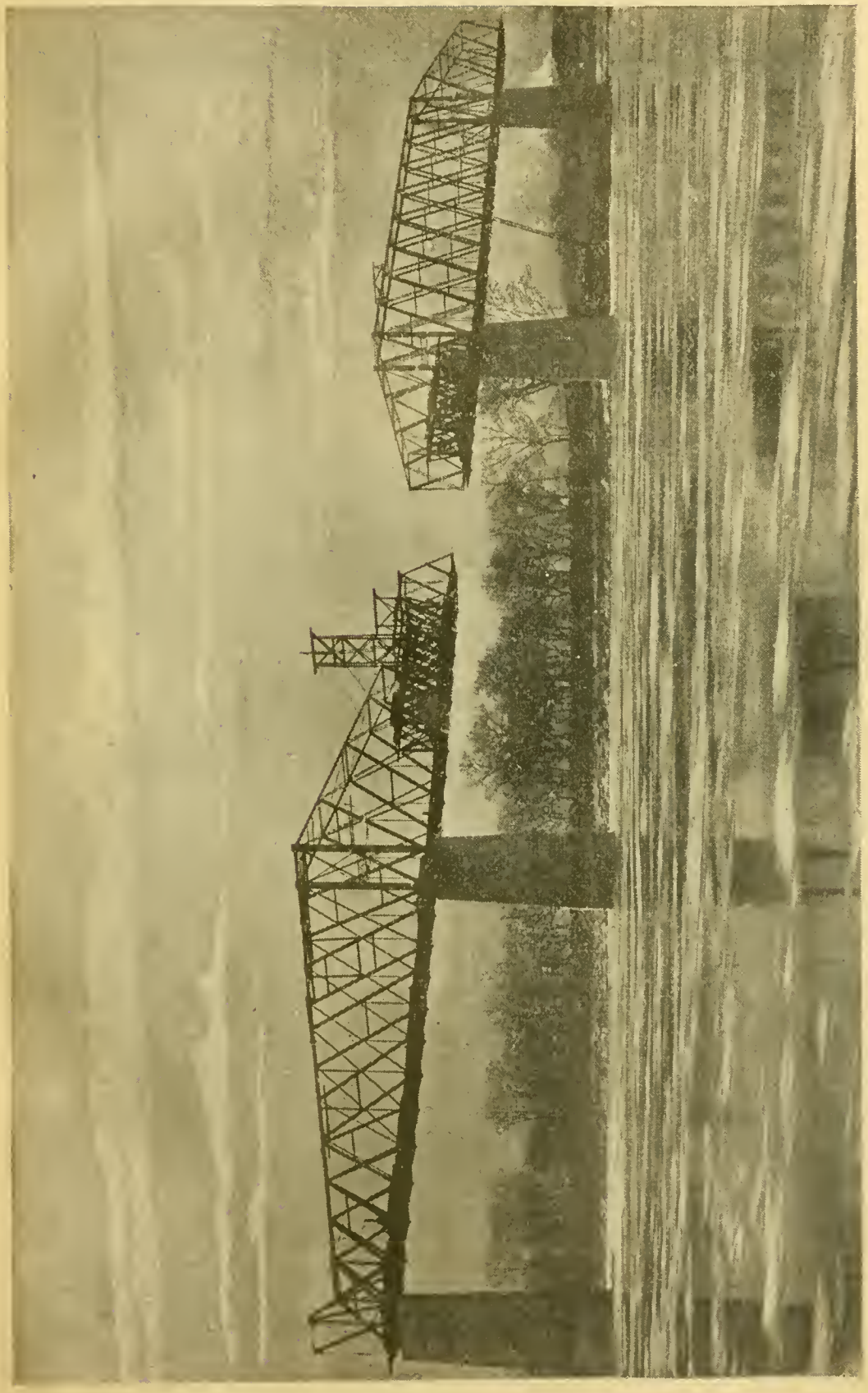


PLATE XXI
TRANS. AM. SOC. CIV. ENG'RS.
VOL XVII, NO. 365.
MOULTON ON
KENTUCKY AND INDIANA BRIDGE.



AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

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(Vol. XVII., October, 1887.)

# DISCUSSION ON THE KENTUCKY AND INDIANA BRIDGE.*

J. W. SCHAUB, M. Am. Soc. C. E.—The paper of Mr. Moulton on the Kentucky and Indiana Bridge is a valuable record as to the results of tests, etc., and it is to be hoped that Mr. Moulton will supplement his paper so as to discuss some of the salient features of this remarkable engineering structure. The novel and intricate systems of cantilevers are the work of the late Mr. C. Shaler Smith, to whom Mr. Moulton gives just tribute. A similar arrangement of cantilever spans are now being built by the Union Bridge Company at Poughkeepsie, N. Y.

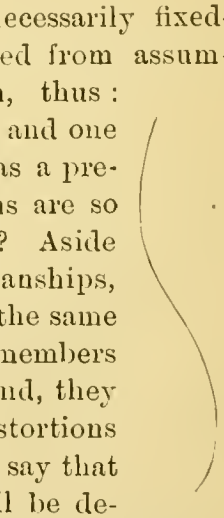
Mr. Moulton says: "No unusual problems presented themselves in the design excepting those natural to the system of web members, and the series of cantilever spans continued to such an unusual extent."

As I am somewhat familiar with the methods of computation employed by Mr. Hemberle, and the details used in the construction of this work, I was naturally disappointed in seeing Mr. Moulton omit a discussion of some very important features. The changes made in Mr. Smith's original plan, as to the style of trusses, depth, etc., possess many advantages as to economy of design; and this, in connection with the use of the Launhardt formula, as employed here, permitting much higher unit strains, made a vast difference as to the final cost of the superstructure. However, it will not be out of place to discuss some of the marked differences in the two designs. Mr. Smith's plan, which provided for a floor riveted between vertical posts, possesses many advantages over a floor suspended from the lower chord. It is only necessary to examine the two kinds of floors when in use to be convinced of this fact. However, in the Kentucky and Indiana Bridge the floor beams are screwed up against the lower chord, making a rigid floor if properly screwed up.

* Transactions, Vol XVII, page 111 (September, 1887). The Kentucky and Indiana Bridge, by MACE MOULTON, M. Am. Soc. C. E.

The use of a tight fitting pin at the intersection of the web members in the trusses was here used ostensibly for the purpose of shortening the compression members by one-half their total length, and considering them as fixed-ended where they are held by the diagonals at their intersection. To be sure this is correct as far as our principles of statics go; but it is easy to see that distortions must take place in the web members themselves, as soon as a load comes on a structure framed in this manner. For example, let us suppose the trusses to be framed with a camber when there is no strain in any member. This is equivalent to laying the trusses flat on the ground. The web members now intersect at some point below where they would intersect if the trusses had no camber. Now, when the load is put on the truss, the chords assume their normal position. The point of intersection must rise until, finally, when the entire camber is taken out of the trusses the members will have assumed their normal positions, provided the web members are not fixed to each other at their point of intersection. Where they are held as in the Kentucky and Indiana Bridge they will only tend to assume their normal positions and consequently distortions must take place. I would like to know from Mr. Moulton, what is the maximum difference in the length of the upper and lower sections of the diagonals in the 360 feet span? Knowing this difference it is easy to calculate the theoretical movement of the intersection of the diagonals during the transition from camber to no camber in the trusses. This difference would represent the total movement that would take place if the intersection were not fixed and the eye-bars had no sag.

Furthermore, a member held at any point is not necessarily fixed-ended at that point unless it is in some way prevented from assuming a line of double curvature under compression, thus:



Is it fair to make a test on a column with one pin end and one square end in a testing machine, and use the results as a precedent in designing a structure, wherein the conditions are so much at variance with those of the testing machine? Aside from the elements such as imperfections in workmanships, vibration and impact, do we approach anywhere near the same conditions as those of the testing machine. First, the members are not shortened by one-half their total length; second, they are not fixed-ended; and last, they are subject to distortions imposed on them by passing loads. The specifications say that the numerator in the column formula (Rankine's) shall be determined by the Launhardt formula, but Mr. Hemberle wisely disarmed criticism in using the largest sectional areas for compression members, obtained either by substituting directly 13 000 in the numerator of the column formula, or by taking the permissible working stresses as determined by the Launhardt formula. However, do the results of the tests on the full sized columns, especially the post made from compres-

sion steel, justify the use of 13 000 pounds in the numerator of the column formula?

Another prominent feature here, which is also a characteristic of several very important structures recently built, is the use of eye-bars in unison with built members to resist alternate tensile and compression strains. In the top chord of the 360 foot span will be found three panels at each end of the span where eye-bars are placed outside of the built chord sections. As the tensile strains here are large and the compression small, it was thought advisable to avoid excessively heavy built members and consequently heavy field riveting at the splices, where eye-bars could be used to advantage; and I believe it was here specified that the eye-bars and corresponding chord sections should be assembled in the shop, to avoid any discrepancies in their lengths. The use of eye-bars and built members in unison to resist tensile strains is very doubtful, especially where it is extended over a great many panels. The built member, owing to its continuity, gets more than its share of the strain in every case I have seen. This is partly owing to the play the eye-bars have on the pins, the bending of the pins themselves, and also to the strains which take place in the heads of the bars, which do not occur in the built member. In a very important structure recently built, this fact is exemplified to its fullest extent in the excessive deflections obtained, which are accountable in no other way. In the Kentucky and Indiana Bridge this combination of eye-bars and built members was used so moderately that it is hardly questionable as to its effects, and could not have been avoided without very much increased cost to the contractors.

The increased unit strains which were allowed by the Launhardt formula is another striking feature. The specifications say: "The maximum and minimum stresses in compression and tension, as found by the before-mentioned loads, with a factor of safety of not less than five, are to be used in determining the permissible working stress in each piece of the structure, according to Launhardt's formula, as follows:

$$a = u\left(1 + \frac{\text{minimum stress in member}}{2 \times \text{maximum stress in member}}\right)$$

in which

Tension is algebraically —
Compression " " +

$a$ = permissible stress per square inch.

For double refined iron in tension (links or rods):

$u$ = ..... 9 000 pounds per square inch.

For double refined steel in tension:

$u$ = .....14 000 " " "

For rolled iron in tension (plates or shapes):

$u$ = ..... 8 500 pounds " "

For rolled steel in tension:

$u =$ .....14 000 pounds per square inch.

For rolled iron in compression:

$u =$ ..... 7 500 " " "

For rolled steel in compression:

$u =$ .....13 000 " " "

The Launhardt formula, as before given, embodies the two formulas given by Weyrauch, inasmuch as the formula becomes, when the minimum and maximum stresses have the same sign + or —.

$$a = u\left(1 + \frac{\text{minimum } B}{2 \times \text{maximum } B}\right)$$

when they have opposite signs

$$a = u\left(1 - \frac{\text{maximum } B^1}{2 \times \text{maximum } B}\right)$$

in which

Max. $B$ = the greatest stress upon the member, whether tension or compression.

Min. $B$ = the least stress of the same kind.

Max. $B^1$ = the greatest stress in the opposite sense.

See paper of Mr. Joseph M. Wilson, M. Am. Soc. C. E., page 391, Vol. XV. However, Weyrauch says that $u$ = ultimate strength per unit of section for any number of repetitions of load. In the Kentucky and Indiana Bridge $u$ = for rolled steel in tension 14 000 × 5 (factor of safety) = 70 000 pounds per square inch, which should be the ultimate strengh for any number of repetitions of load. What do the tests on full-sized eye-bars show? The ultimate strength of 12 bars tested to destruction varied from 63 500 to 78 500 pounds per square inch.

I presume the bars were tested in the usual manner, which is practically a single application of the load. Judging from the results of the tests, were the values of $u$ here properly taken?

In the 267-foot skew cantilever span we find in one case a working stress of 17 500 pounds per square inch in tension. Granting the ultimate strength of large bars at 70 000 pounds, we obtain a factor of safety of four for dead load and live load combined. Here the ratio of dead load to live load strain = 1. Therefore, if 5 be the factor of safety for live load, the factor of safety for dead load must be 3. Or if 6 be the factor for live load, the factor for dead load must be 2, and so on.

The aim of the Launhardt formula is certainly very scientific, but would it not be simpler to fix upon a factor of safety for dead load, and a factor of safety for live load, obtain separately the sections required for each, and add them together for a final result? This would be very simple for members subject to only one kind of strain. For members subject to alternate strains we could use a certain percentage of the lesser strain and add it to the greater for the resultant live load strain. As

the Launhardt formula requires the maximum and minimum strains to be determined for each member, it would seem that in an ordinary discontinuous span the dead load strains would be the minimum. However, it is easy to see that for the web members the counter-strains would tend to reduce the dead load strains, as the live load is backing off the bridge. This necessitates finding the counter-strains for all web members and combining them with the dead load strains to obtain the minimum strains for web members. Does a refinement like this seem rational? Would it not be simpler to say, for example, the factor of safety for live load shall be 6 and the factor for dead load shall be 3, and the sections for each shall be added for a final result?

The specifications required steel for compression at 80 000 pounds ultimate, and steel for tension at 70 000 pounds ultimate. Would it not be advisable to use the same material for both tension and compression, especially where members are subject to alternate strains. In the Lachine bridge, recently completed, no distinction was made, and a uniform steel at 60 000 pounds ultimate was used throughout.

The tests made by Mr. Moulton as to the effects of shearing and hammering steel plates are very interesting, and tend to establish a valuable precedent. The specifications required all holes to be punched and then reamed one-eighth inch larger, leaving the holes of the proper diameter as required by the size of the rivet. As the ultimate strength for steel is gradually being reduced, it is to be hoped that practice will eventually establish a uniform steel at 60 000 pounds ultimate, and avoid the expensive processes of planing the edges of sheared plates and reaming holes for rivets.

Palmer C. Ricketts, M. Am. Soc. C. E.—The difference between the allowed shearing resistance of steel and iron in the specifications under which this bridge was built, 10 000 and 7 000 pounds per square inch, seems to be too great. This is here referred to because it is believed that the difference usually allowed in favor of steel, about 33 per cent., is greater than the conditions existing in steel and iron structures and the strength of the material warrant.

Although the Watertown experiments seem to show generally that this amount may be allowed, most of them were made with single riveted quarter-inch plates, with smaller pitch and rivets than is generally permissible in bridge practice; and if the difference in resistance of either metal in punched and drilled joints be considered, the few directly comparable experiments on double riveted joints with thicker plates made there reduces this percentage appreciably.

The results of English experimenters bear out this inference and would seem to indicate that if 7 500 pounds per square inch be taken for iron rivets in shear, the amount allowed for steel should be more nearly 9 000 than 10 000 pounds.

As far as failure by nearly pure shear is concerned, experiments of the

writer, published in the Transactions of this Society for March, 1887, on iron and steel, three quarter inch round rivet material, some of the latter taken from lots used for rivets in the bridge which is the subject of this paper, show that the resistance of steel of 0.10 per cent. to 0.14 per cent. carbon and about 0.40 per cent. manganese, with 58 000 to 64 000 pounds ultimate tensile intensity, is not more than 15 per cent. greater than that of iron. The considerable difference in resistance when the cutting edges are sharp and rounded, is also shown there.

It is interesting to notice that if in the results of the tests of the two ong columns given on pages 160 and 161 we disregard the strain for the stress on the column below 20 000 pounds, regarding this amount as necessary to begin a somewhat uniform strain in the central measured length which the first compressive increments in the tables show to be the case, we get, by using the mean of the remaining strain increments up to 14 000 pounds per square inch, beyond which the column would not be liable to be stressed, co-efficients of elasticity of 27 200 000 and 28 600 000, and for stresses up to 30 000 pounds per square inch 26 500 000 and 27 400 000.

The mean compressive co-efficient for the eight built steel posts given by Mr. J. G. Dagron, M. Am. Soc. C. E., in his paper read at the Annual Convention in July, 1887, had an average value of 27 600 000, varying from 25 800 000 to 29 500 000.

In these experiments it is seen that the value of this quantity for these built pieces of structural steel, stressed as they would be in place, is not very much less than its average value, about 29 000 000, determined by experiments on small pieces.

A consideration of the tests of the eye-bars given in this paper, compared with those of the three-quarter inch rounds from the test ingots, together with a comparison of other results of the same kind from some Watertown experiments, shows that the difference in elastic limit in favor of the metal from the test ingot varies in these cases from 10 to 33 per cent., and in ultimate resistance from 0 to about 15 per cent., in one case, reaching 26 per cent. The elastic co-efficients seem to remain practically the same.

Samuel Tobias Wagner, M. Am. Soc. C. E.—Among the extremely interesting and well-presented features of Mr. Moulton's paper is the record of tests where comparison is made between those made on a ¾-inch round bar and those cut from the section as finally rolled, and again is this comparison more beautifully shown where the tests are given on a ¾-inch round on a specimen cut from the bar, and on a full-sized manufactured eye-bar. If in steel a closely approximate relation can be established between tests prepared on ¾-inch round bars and several sizes of rolled shapes, plates and bars, all from the same heat, so that upon the ¾-inch round tests the heat could be at once accepted or rejected; the time saved in the completion of such a contract as Mr. Moulton's paper de-

scribes, would be considerable, and the satisfaction to both inspectors and manufacturers would be of great value.

Some four years ago this point was presented very forcibly to the writer at a prominent steel works in Pennsylvania, while he was acting as inspector under some rather poorly prepared specifications, where, for want of some sufficiently prepared and discussed matter on this subject, the test pieces to determine the acceptance of the heats were prepared from plates rolled to certain widths and thicknesses in order to give on them approximately the same amount of work as the final section would have, and no tests were made on a ¾-inch round, although the steel makers claimed that the results obtained from these tests would be satisfactory. The material was a 60 000-pound steel, with a required elongation of 23 per cent. in 8 inches, and had to be accepted from the steel mills in the shape of blooms, ranging in size from 3 by 3 to 7 by 10 inches, and which were to be rolled down at another mill into sizes varying from 1½ by 1½-inch L to 9-inch deck beams. Upon these the final tests were to be made under a very rigid inspection. It became necessary therefore to have several different sizes of rolled specimens from which to cut tests in order even to approximate the amount of work and reduction from the ingot.

From a careful examination of Mr. Moulton's results on ten different heats, I have selected some eighteen different characteristic tests, and striking a rough average from these, I find:

*First.*—The elastic limit is higher in the ¾-inch round than the other tests, by about 2 100 pounds in thirteen tests, while it falls to about 1 100 pounds below in five others.

*Second.*—The ultimate strength shows in fourteen tests a greater strength in the plate specimens than in the ¾-inch round, averaging 2 300 pounds, while in four tests the ¾-inch round shows a greater average strength of 3 100 pounds.

*Third.*—The elongation is decidedly the least fluctuating, and always shows greater in the ¾-inch round than in the plate test, but the average in the eighteen tests is only about 2.4 per cent. higher.

Mr. Moulton's results are extremely interesting and valuable, and agree with what experience I have had and results I have seen on the subject, although his are by far the most complete. It is to be hoped that some one else will give us more data on this subject, so that these results may be confirmed, although, I believe, for steels from say 60 000 to 85 000 pounds per square inch ultimate strength, the fluctuations will coincide very closely with the results shown here. Mr. Moulton remarks that the specifications called for the tests to be made on a ¾-inch round, and this is an encouragement for its future use, where the check tests are as good as these are.

The results obtained from the 7 by 7 inch square ingots, I think, are unusually good for such a severe method of testing the quality of the

material, although I am glad to see they were abandoned, as certainly the danger of unjust results from them is very great.

The matter of cast-steel clevises is one of great interest to bridge builders, as it is a step toward the substitution of steel castings for what are now expensive forgings. The results obtained are not surprising, as there seems to be very great difficulties attendant upon the manufacture of small steel castings, but it is to be hoped that the steel makers will soon solve the problem. It would be of interest to know under how great a head of metal these clevises were poured, and in what position they were moulded.

If in the discussion Mr. Moulton could tell us something about the method of working the steel in the shops, that is, whether it was drilled or punched and reamed, or simply punched, and also whether special caution was taken to avoid sharp corners and edges, it would add a few more interesting items to his already complete description.

Joseph M. Wilson, M. Am. Soc. C. E.—I should say that the moving train load assumed for this bridge, in view of modern service requirements, or even those of ten or more years ago, was too little. After considering carefully the lengths of spans, I should not have made it less than 3 000 pounds per foot, and perhaps more.

The formula adopted for the maximum and minimum stresses in compression and tension appears to be for stresses only of one kind (all compression or all tension). No modification is made for stresses acting in opposite directions, although from the type of the structure I should judge that such stresses did occur. I hardly understand why the paper should make use of the term "factor of safety of not less than five" in connection with the use of this formula. The stresses introduced into the formula for the various cases must evidently be the actual stresses, not these stresses divided by five, and the value of $u$ is assumed at various quantities for different materials so as to give the permissible stress $a$ per square inch. While the process of calculation may have been correctly followed out, the explanation of the use of the formula is certainly not clear. Also, on page 136, there appears to be a misprint. It should be:

$a$ = permissible stress per square inch.

$u$ = for double refined iron in tension (links or rods) 9 000, etc.

$u$ is here, as I take it, an assumed permissible stress per unit of section for any number of repetitions of load, adopted from experience or precedent, and not by any means one-fifth of a supposed ultimate strength for the material in question.

The values of $u$ given in this paper for iron are the same as I used some years ago with the same formula, although I also used Weyrauch's second formula for pieces subjected to stresses acting in opposite directions. These values were printed in specifications of mine, issued for private use only in connection with the Pennsylvania Railroad. The

formulas and values of $u$ were afterwards modified to those given in my published specifications (see Transactions Am. Soc. C. E., Vol. XV, June, 1886), as presenting in my view an improved practice.

The formulas for reduction of members in compression vary slightly from those which have been used by myself. The question of requiring the lines of the neutral axis of all pieces meeting at a joint to meet in the same point, or the placing of pins where used, in the neutral axis, and if not so arranged, the necessity of considering the bending stresses so produced, seems to be ignored.

In the provision for wind strains and vibration, no computation seems to be made for the case of a moving train. The results with a moving train will be different from those of a static load for the whole length of the span.

The method of computing the shearing stress on girders with stiffeners differs from my own theory on the subject (see Transactions Am. Soc. C. E., Vol. XV, June, 1886).

The requirements of the specification for iron and timber and for workmanship are very meagre. The mere statement that they should be "as usual in first-class work" would seem to me hardly sufficient in a "first-class" specification.

I regret that Mr. Moulton has not given us a summary of what practical inferences he would gather from the results of his tests as shown in his tables. Of course any one can by an examination of these tables draw inferences, but I would rather have seen his own conclusions.

On page 163 the drawing of clevis appears to be inaccurate. The connections of the arms with the nut are evidently not correct.

Concerning the question of adopting tests from the ingot or the finished material, for the acceptance of the latter, it would appear from the results on pages 165–6 that ingot tests give higher results than finished material tests. If the former are adopted, as seems to be urged upon engineers by manufacturers, an allowance should be made for subsequent deterioration, which might be some established percentage.

C. L. Strobel, M. Am. Soc. C. E.—A record of the construction of a large bridge in such detail as presented by Mr. Moulton is very interesting and valuable, and he merits our thanks for his labor, which has been very considerable.

Mr. Moulton's experience with cast-steel agrees with my own. In the present stage of its manufacture, and in all probability for a long time to come, this material is not suitable for use in bridges when subjected to tension, because its strength is too uncertain. I have, however, used it for such parts as bolsters, shoes and pier-boxes, to great advantage. These parts are mainly in compression, massive in size, and the stresses are low, so that there is ample margin for safety even if honeycombing or the other defects to which this material is subject exist. Cast-steel has the great advantage over cast-iron, that, if a suit-

able grade is selected, and it is properly annealed, it is tough and malleable. It does not have the brittleness of cast-iron, and it has the advantage over wrought-iron, that it can be cast into massive and complicated forms, which it would be impossible to make by riveting together plates and angles and obtain equal efficiency.

The results obtained with sheared and planed steel plates indicate simply less elongation for specimens having one edge sheared and the other planed, as compared with specimens having both edges planed, the ultimate strength remaining about the same. A marked difference would have been found, however, in the ultimate strength and other qualities of the specimens had they been subjected to vibratory action while under stress. A slight crack, rent, or a sharp indenture on the edge of a bar or plate is a point of weakness in steel greater than in wrought-iron (provided the direction of the break in the latter case is across the fiber) for the reason that steel is a homogeneous material and readily yields to new conditions, so that incipient fracture acts like an entering wedge, and failure gradually takes place by an accretion of small increments of fracture. One of the striking features of the behavior of steel columns when tested to their ultimate strength is their sudden collapse soon after the elastic limit has been exceeded. The metal bulges between rivets or other point of support, flowing away from the line of stress suddenly and freely, therein acting very differently from the behavior of wrought-iron under similar conditions. The latter retains its form longer after the elastic limit has been exceeded, and is more apt to fail by bending as a whole; or, if it gives way by bulging or buckling locally, it does not do so as freely nor as suddenly as steel.

I should like to ask Mr. Moulton what object there was in using pine for the upper layer of plank in the wagon way, and oak for the lower layer placed diagonally? The reverse is the usual practice; the lower layer is made of pine placed at right angles to the joists so as to make the span between them as small as possible, and the upper layer is made of oak so as to have a hard material as the wearing surface, and the plank in this layer is placed diagonally or lengthwise of the bridge. I should also like to ask whether the floor-beams are without connection to the lower chords, as drawn on Plate XIX, in which case it is difficult to see that there is any chord to the lateral system.

In both the designs of superstructure as adopted and as originally proposed, double systems of triangulation are used, which give rise to a certain amount of ambiguity in the stresses not desirable though not seriously objectionable. How much of the weight of the suspended span in the cantilever system is carried by one and how much by the other of the two systems of triangulation, is dependent upon their relative deflection and the accuracy of length and fit of their parts. The design adopted is certainly more economical of material than the other, and it has the further advantage that the chord lines are straight be-

tween the apexes of each system (not broken as in the top chord of the other design), whereby one of the elements of abiguity existing in the latter is eliminated.

Henry B. Seaman, M. Am. Soc. C. E.—If it is at all feasible to use steel rods with screw ends, the precaution of a soft steel, with fillets at the bottom of the threads, would seem to make it so; but even then a roughly cut thread is unavoidable occasionally, and is an element of weakness.

The real test for the rods will be in the bridge, where they are subjected to a repetition of strain and to vibration, rather than in a testing machine, where the strain is continuous; as vibratory strains will more effectually develop cracks.

The fact that the upset ends are of greater diameter than the body of the bar, tends to relieve the screws from the effect of impact and thus to prolong the life of the bar; but experience has shown that with ordinary threads, this relief is not always sufficient to save the rods, as they have broken in the thread, showing in the fracture rings of rust, and indicating that the bar was broken in increments.

Mr. Moulton's tests of annealed specimens show in every case a deterioration by annealing. The elongation is reduced, while the reduction of area at the point of fracture is increased, indicating a more plastic but less tenacious material. This is probably more marked in a small test specimen than it would be in a finished bar, but would also occur in the latter to some degree. It is probably due to decarbonization, either by being kept too long at the refining heat, or being cooled too slowly, possibly in the air. When we consider the great influence heat has upon the quality of steel, it would appear advisable to anneal with every lot of bars a specimen of the same material, of full section of the bars, and about 18 inches long, which may be afterwards tested for over annealing; the test is inexpensive and easily made.

There is a clause in the specifications for superstructure which provides that "rivets will not be used where they may be subjected to tensile strain." This is not an uncommon specification, but if it were strictly enforced, it would throw out the connection for lateral bracing shown on Plate XIX. Rivets are preferable in shear on account of the additional resistance of friction; but where tension is necessary, as is shown here, I think it is better to specify some limiting tensile strain.

Rivets are made of the best quality of material, and when shop-driven, the shank of the rivet undergoes less "punishment" than an ordinary upset rod. If sufficiently strained, rivets will work loose under shear, as well as under tension; and if rivet heads of hard steel drop off, it would seem to be the fault of the material rather than the kind of strain, for they drop off under either strain.

I do not wish to be understood as advocating tension on rivets. It should be avoided wherever possible. But I do not think it such a

"bugbear," as is often considered. I think the tensile value of a rivet should be considered a function of the shearing value.

C. E. Emery, M. Am. Soc. C. E.—The ultimate tensile strength of steel boiler plates for the United States Navy and the better class of marine works has for a number of years been limited to 60 000 pounds; a latitude of from one to one and one-half thousand pounds above and below that figure having however been allowed to meet practical conditions at the mills. Substantially these limits have been fixed also for the steel used in the new naval cruisers. Even with steel as low as this, initial strains are introduced by punching and shearing which materially reduce the strength of the plates, though the difficulty may be practically overcome by reaming the holes and planing the edges.

It is believed that a reaction will soon set in in regard to increasing so largely the sections of members subject to reverse strains. The question is very materially modified by the periods of rest available between the applications of strains. In the experiments on which the original formulas were founded, the reverse strains necessarily followed each other in comparatively rapid succession in order to get results in a reasonable time, and moreover the applications were kept up continuously until failure resulted or particular experiments were abandoned. Had there been periods of rest after the application of reverse strains for a time, the molecules would have re-established their equilibrium and the material been as good as before. My recollection is that in the experiments it was necessary to carry the strains nearly up to the elastic limit of the material to produce rupture under conditions of continuous application. Bridge members cannot receive such severe tests when the reverse strains are well within the elastic limit, the periods of rest between maximum loads allowing the strength to become restored without fail. This is proved by the life of iron and steel rails, which even on a bad foundation remain intact under reverse strains very much greater than ever occur in any other form of construction.

Mr. Theodore Cooper.—Allow me to ask, did I understand you to say that your experience was to ream the holes in steel?

Mr. Emery.—In the construction of boilers we permit the builders to punch the holes in steel of 60 000 pounds tensile strength and afterwards ream them.

Mr. Cooper.—Do you recommend it?

Mr. Emery.—I do not recommend it. I would rather see the holes drilled. I can only say that it seems a necessity to permit it, as plenty of boilers are made from steel in the same way as iron without even reaming the holes. The method of reaming seems to answer under practical conditions and therefore under the circumstances is allowed. Steel is, however, so much abused in boilers that I sympathize strongly with those who will not permit its use at all.

THEODORE COOPER, M. Am. Soc. C. E.—I am rather pleased with Mr. Schaub's remarks in regard to the Launhardt formula, but he don't go far enough. What are Launhardt and other similar formulas based upon? In all the discussions which have recently taken place in regard to the use of this form of formula I have found no evidence that the writers have studied the original experiments from which this form of formula has been derived. The discussion has not been upon the merit of the form but rather upon the proper constant 1, 2 or 3 to be inserted before the ratio of the minimum and maximum stresses.

The very interesting experiments made by Wohler in 1867, and first published in English in *Engineering* of 1871, are the basis upon which these formulas are built.

Wohler extended his tests over several years, and submitted some of his test pieces to as high as 132 000 000 applications of the strain. He made some of his tests by direct tension, but the largest number were made by tensional and bending tests very ingeniously arranged so as to confine the tests within limited ranges. While he formulates his results in strain per unit of area, any one must recognize that these fiber strains would, under any method of calculation, be very doubtful in their amount. After years of investigation he was, like every true experimenter, very cautious in drawing any deductions. He does state that "variations of strain may with equal security take place within the following limits, it being of course assumed that in all cases the maximum strain is less than that required to produce fracture under a static load."

For iron subjected to tension and compression.

| | | | |
|---|---|---|---|
| — 17.120 | pounds to | + 17.120 | pounds |
| 0 | " | + 35.310 | " |
| + 25.680 | " | + 47.080 | " |

etc. for other cases.

The discovery of this apparent anomalous action of a metal which had an elastic limit, as result determined, of 35 000 pounds, breaking when subjected to strains not exceeding one-half of this amount, led to the absurd and unscientific term "fatigue of metals." This result was totally at variance with the acceptance of the perfect elasticity of the metals as heretofore accepted.

A careful study of the original experiments led me at that time to the belief that the true explanation of this so-called fatigue of metals was in the idea that this iron did have an elastic limit somewhere near 35 000 pounds, but that it must be measured not from the zero point of strain each way, but between the extreme strains, algebraically considered. I have also believed that what some experimenters claim to be a raising of the elastic limit of a bar by strain is simply a shifting of this

elastic range. If this belief is the true one, this whole doctrine of the fatigue of metals falls to the ground, and with it all those theoretical formulas deduced from such a theory.

Another argument made by the authors for this form of formula is the old American practice of allowing a larger "factor of safety" for live loads than for dead loads.

A careful examination of the evolution of the original Launhardt formula will show that the tendency is fast bringing the formula back to an identical form with the old American practice, to allow double the unit strain for dead loads that we do for live.

This is in accordance with my own views and the principle I have adopted in my recent bridge specifications. The reason for this practice is not, however, based upon any faith in the fatigue of metals, but upon the relative uncertainty of the loads and the strains induced. We can very exactly determine the dead load of a structure, and can have considerable faith in the strains determined therefrom; but for the live load there is much uncertainty as to its amount, and also as to the strains induced by this live load in rapid motion over a structure. We are, therefore, justified in accepting a "factor of ignorance," and with one-half of the allowed strain for dead loads.*

I am just reminded by a member that I have not made myself plain in regard to the reversal of strains in a bridge structure.

As long as we do not work our material by any manner of stress beyond the range of elasticity, I do not consider it can be injured, but of course we still have the doubt in regard to our live loads and their action. We should therefore keep well within the actual range in proportion to our ignorance of the absolute strains produced upon any members of the structure.

The reversal of strains therefore cannot hurt a bar with this proviso.

I do consider it very important to provide for extra strong con-

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* NOTE (added by letter).—Since making the above remarks, I find in *Engineering*, November 25, 1887, the following: "Professor Bauschinger has made a series of experiments on the alternation of the elastic limit, the experiments extending over several years. He concludes that "the elastic limit in tension is in general very different from that of the material when subjected to compression, and artificially raising the elastic limit in tension causes the limit in compression to be decreased, and this may even pass through the point of zero stress. In other words, a bar of steel or iron has two elastic limits, and whatever position these occupy on the scale of loads, the range between them is nearly a constant quantity. By alternately stressing a bar in tension and compression just beyond the elastic limits, these after a certain number of repetitions occupied positions equally distant from the point of zero load, and the limits thus obtained are called by Bauschinger the natural elastic limits of the bar. It was then noted that the stress corresponding to these limits sensibly coincided with that found by Wohler as the limiting stress to which a bar could be subjected to alternate tension and compression. It would thus appear that a bar will bear an indefinite number of repetitions of stress, provided the range of stress does not exceed the elastic range mentioned above."

This is a very strong confirmation of the views above expressed, and which I have held for more than fifteen years.

nections where the strains can be reversed. The alternate action upon the pins, rivets and other connections, are much more injurious in comparison to the same equivalent strain in one direction. In rivet connections, owing to this alternate action tending to work them loose, I think we should allow a greater number than where they are only pulled in one direction. The same would also apply to other connections where there will be more or less lost motion.

Mr. Emery.—Mr. Cooper's remarks recall my original reading of the experiments. My recollection at present—and I would like to be corrected if wrong—is that they did not succeed in breaking a member, under reverse strains, until it was loaded by the method of computation up to or beyond the elastic limit. The ridiculousness of formulas extending within elastic limit, when based upon experiments without, is warranted by the remarks of Mr. Cooper. His explanations coincide with my remarks in regard to the intermittent application of reverse strains. When there is time for the metal to rest it assumes its former state; so, even if it be strained up to the elastic limit in tension and then be allowed to rest, it can safely afterward be carried to the elastic limit in compression. When, however, the strains repeatedly alternate from plus to minus, the algebraic difference must be taken, giving the numerical sum of the two, and the evidence seems to be that such sum cannot, for continued repetitions of strain, ever exceed the difference between zero and the elastic limit in either direction; but periods of rest between the intermediate applications of reverse strains change the rule entirely. The effect of flexing metals beyond the limit of elasticity, without rest, is readily seen with a material like brass, which hardens each time it is so bent. It is well known that such material cannot be bent back and forth in the same place many times without breaking. The rails of railroads, however, which must at times be strained as high on the outer fibers as 25 000 pounds per square inch in each direction, under the passage of a heavy locomotive on a yielding road-bed, rarely break, simply because the material has a period of rest between the passage of trains, and, at some hours of the day, very long periods of rest, so that on the whole the rails periodically resume their normal condition and are prepared for the next series of abuses.

It is well known that we must prevent motion. As soon as the particles are allowed to move, the jars may bring the strains beyond the limit of elasticity and an actual set take place, which, after a series of repetitions, will cause failure. Persons here present will recollect that in one of Mr. Metcalf's papers it is stated that different materials were tried for the connecting rods in a machine running at a high speed in which the weight of the parts brought a bending action on the rods. It was found eventually that high steel stood much better than any other material, the reason being that it was stiff enough to prevent the actual motion of particles, and there being no sudden jars, there was no danger

of rupture. It should be borne in mind that the strains on the outer fibers of bars subject to transverse strains are never as high as shown by formulas, for the reason that the fibers between the same and the neutral axis can, on account of the bending of the bar, take up a greater portion of the load than shown by the customary formulas, on the principle set forth in my paper several years ago on "The Transverse Resistance of Materials.

Wm. H. Burr, M. Am. Soc. C. E.—Aside from the general character of a work of this magnitude, the excellence of its general design and the skill and energy displayed in the erection of these parts requiring false work over a river subject to great and violent changes of stage during the most dangerous season of the year, all of which reflect great credit on the engineers and contractors for the work, those portions of the paper outlining the results of special treatment of the steel members at different stages of their fabrication are of unusual interest and importance at the present time.

The conclusions to be drawn from the regular tests are only such as are usually reached in the ordinary testing of the material for steel plates and angles. The ingot tests of $\frac{3}{4}$-inch rounds do not give results essentially different in any respect from those of specimen tests of the rolled material, provided always that the amount of work expended on the material is essentially the same in the two cases.

The elastic limit runs a little over six-tenths the ultimate, and the final contraction is roughly twice the final stretch, approximate averages being considered. The exceptions to the latter ratios confirm the general principle preceding, since they occur in connection with the tests of eye-bar steel. In the case of this material, the steel of the bars naturally received much less work than that of the $\frac{3}{4}$-inch rounds; hence the elastic limit of the latter in general considerably exceeds that of the former, while the ultimate resistances are not very different. The differences between the elastic limits decrease as the bars become smaller and thinner.

The final contractions and stretches are about the same for the $\frac{3}{4}$-inch rounds as for the specimens from the bars. The effects of annealing are well shown in the tests of full sized eye-bars, which give an appreciable although not great decrease of elastic limit and ultimate resistance below the values yielded by the specimens taken from the bars before annealing. The final reduction is but little if any below that of the test specimens, except in one or two special cases, but the final stretch is of course much less.

The special tests, however, on shearing and cold hammering, and incidentally those on steel cast clevises, are of more than ordinary interest and importance. In noting the effect of these operations the grade of steel used is to be particularly observed, for there are good reasons to believe that the operations indicated, as well as some others

incidental to the production of finished bridge members, and which are usually excluded by specifications, may with propriety be employed with mild structural steel used under compression, but should be carefully avoided with a higher grade of metal, however it may be used.

In this paper, for example, on pages 165 and 166, the sheared edge specimens with edges not planed belonged to metal varying from 66 000 to 75 000 pounds in ultimate tensile resistance, yet the elastic limit and ultimate resistance were not appreciably affected. The final stretch and reduction, however, were reduced 35 to 50 per cent. This would be a matter of little consequence in a compression member, but would scarcely be permissible in tension. In 60 000 pound steel the effect of the shears would be still less, while in the 80 000 pound metal used in this structure it would be materially increased and could not be ignored.

The effects of hammering steel plates, as shown on page 167, are far less than we are led to believe would result, if we observe the great care with which hammering is excluded by specifications from shop processes. In fact, it is very difficult to draw from these tests the evidence of any deleterious effects resulting from the hammering of specimens. Some were hammered while hot and then cooled in ashes; others were hammered while cold, some of them being subsequently annealed, and the remainder allowed to remain as left by the hammer.

Those hammered hot and cooled in ashes seemed to gain slightly in ultimate resistance and lose a very little in final stretch, while those hammered cold and untreated subsequently seemed to lose a little in elastic limit, which was a rather remarkable result.

The sum total of the twelve tests does not reveal any essential effect of the hammering, nor anything that could not properly be ignored in any considerations affecting structural design.

Nor could any appreciable effect of the hammer be discovered in the tests recorded on page 168. The annealing clearly reduced the elastic and ultimate resistances and increased the ductility, but it is impossible to detect any influence of the hammer.

Some recent experiences of the writer also point strongly in the same direction as the preceding tests, and confirm the opinion that mild structural steel may be hammered cold with impunity under easily fulfilled conditions. I had a $\frac{9}{16}$-inch plate 5 feet 9 inches long and 12 inches wide severely hammered with heavy sledges in the hands of three men. These men, operating together, pounded the middle of the plate from fifteen to twenty minutes, until that portion of it was completely covered and indented with hammer-marks and the plate very much bent out of its original plane shape. The end portions, however, were in their normal condition. The plate was now straightened cold and six strips 18 x $\frac{9}{16}$ x $2\frac{3}{8}$ inches cut from it. Two of these were from the end portions, untouched by the hammer and in a perfectly normal condition.

The other four were cut from the central portion of the hammered part and had sustained the heaviest hammering, so far, at least, as could be determined from the marks and indentations. Four of these pieces were then pulled apart in a testing machine with the results given for Nos. 1, 2, 3 and 4 in the following table:

RESULTS OF TESTS OF UNANNEALED STRIPS OF $\frac{9}{16}$-INCH STEEL PLATES.

*All Test Pieces* $18 \times 2\frac{3}{8} \times \frac{9}{16}$ *inches.*

| No. | Pounds per Sq. In. at | | Per Cent. of Final | | Remarks. |
|---|---|---|---|---|---|
| | Elac. Lim. | Ultimate. | Stretch in 6-in. | Contraction | |
| 1. Normal..... | 39,200 | 68,500 | 33 | 53 | |
| 2. do ..... | 40,800 | 70,000 | 31 | 35 | |
| 3. Hammered..... | 53,100 | 71,500 | 16 | 41 | Failed outside of gauge marks. |
| 4. do ..... | 46,200* | 71,100 | 9 | 48 | do do do |
| 5. Hammered twice. | 58,460 | 73,100 | 2 | 42 | do do do |
| 6. do do .. | 58,460 | 73,500 | 3 | 44 | do do do |

*The elastic limit of No. 4 is shown too low in consequence of an accident to the machine at this point of the test. These elastic limits are not well defined for the hammered specimens and indicate only the points at which the scale beam ceased to be quite stiffly held off its rest.

Specimens Nos. 5 and 6 were then taken back to the anvil for a second severe hammering cold with the sledges, and their subsequent appearance showed that they had been most thoroughly abused. In some places the thickness was reduced nearly $\frac{1}{16}$-inch and numerous indentations were of the most violent character. The results of pulling these in the testing machine are shown in the table as Nos. 5 and 6.

Now the results of these tests show that the cold hammering without subsequent annealing materially raises the elastic limit and slightly the ultimate resistance, but decreases the ductility. The amount of the latter effect, however, is uncertain, as failure in every hammered specimen took place outside the gauge marks, between which the stretch was measured. In all cases the contraction is well up to that of the normal specimens. As near as could be observed the specimens were a little less marked by the hammers near one end at which failure took place, showing that the metal flowed easiest where it was hammered least.

The intensified effects of the second hammering are easily seen by the increased elastic and ultimate resistances in Nos. 5 and 6. But all the unreasonable torture to which these specimens were subjected did not at all injure it for duty in a compression member, if, indeed, it has

reduced its tensile capacity. This hammering was done with ordinary sledges, but in no case was a crack started, and that fact undoubtedly explains the process of the tests. If steel plates are straightened cold, however, it should always be done with a "flatter," in order to avoid the possibility of starting any crack.

I also had some double counter-sunk steel rivets driven so as to produce the plates shown in the adjoining sketches. Two plates were riveted together with $\frac{15}{16}$-inch steel rivets, as shown in Fig. 1, with counter-sunk heads on both sides.

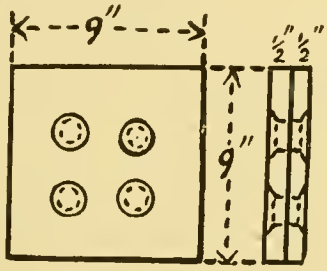


Fig. 1.

Two joints of 3 by $\frac{11}{16}$-inch plates, with $\frac{15}{16}$-inch double counter-sunk steel rivets, were also made, as shown in Fig. 2.

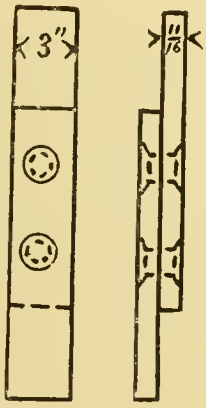


Fig. 2.

All these counter-sunk heads were driven by hand and hammered until and after they were completely black, or, as the workmen say, "until there was no heat in them."

The rivets shown in Fig. 1 were then driven out by placing against the driven head a hammer with a round nose less in diameter than the shaft of the rivet, and sledging it until the rivet yielded. Persistent and heavy blows given by two men striking continuously for about ten

minutes were required for each of three of the rivets, and the fourth one was acting similarly when the operation, about half completed, was suspended for the purpose of chipping off the head in order to discover in this manner, if possible, any evidence of brittleness. The heads of the rivets driven out presented the appearance in the sketch, Fig. 3.

*Fig. 3.*

The round nose of the hammer sheared its way through the head and forced a purely tensile failure in the ring around it. Every rivet head showed extraordinary toughness, and not the slightest evidence of brittle, granular or crystalized material, and developed a resistance far beyond that of the best wrought-iron.

The fourth head showed in chipping just as much toughness and freedom from granular character as the others.

The joints shown in Fig. 2 were pulled apart in a testing machine. The first failed by shearing the rivets at about 57 000 pounds per square inch of the original $\frac{15}{16}$-inch section. The second broke in detail at a total stress of 68 000 pounds, by first shearing one rivet and then pulling the head off the other, in consequence of the large leverage afforded by the thickness of the plates, very much in the manner shown in Fig. 3, if the central case were not punched out. The fracture was wholly a tensile one, and purely silky in character. In none of these cases was there the slightest evidence of a granular character or of any deterioration whatever from hammering the rivet in driving until and after it was black.

While the number of these special tests is not large, they indicate, as do those of Mr. Moulton's paper, that mild structural steel will sustain far more torture cold without injury than is frequently supposed, provided always that no cracks are formed in the manipulation, and these experiences show clearly that they are not easily so formed and may be readily avoided.

F. W. Skinner, M. Am. Soc. C. E.—There is one point in the paper regarding which I have a record that may be of interest.

As I remember the paper, an objection made was, that in proportioning the lengths of main diagonals for camber an objectionable difference would arise between the lengths (normally equal) into which the middle pin divides the members.

A parallel instance is that of the St. John's Railroad cantilever bridge in New Brunswick, built by the Dominion Bridge Company, Montreal, P. Q., in whose service I had charge of computations and drawings. I

don't remember the exact dimensions, but there were two unequal armed cantilevers of six and eight panels respectively and a 100-foot center truss, making a total span of about 475 feet; it was a through bridge with towers 60 and 80 feet high, cantilever diagonals, double intersection.

A camber of 6 inches at ends of cantilever arms under dead load was required, provided it came within certain limits when under train load. The deflections due to distortion of members were figured, and then the decrements calculated that would give the members, when distorted by the given load, the required camber length.

The distortion calculations were based on values of moduli of elasticity obtained from tests especially made upon the steel used, those in compression from full sized columns tested at Watertown Arsenal, and those for tension from small pieces broken at the works, both on Emery machines.

The maximum difference between the lengths of bars of the same diagonal was certainly not above $\frac{1}{16}$ inch, and I have a very strong impression that it was almost exactly $\frac{1}{32}$ inch; at any rate it was decided to neglect it in the shop, and the work went together entirely satisfactorily and camber did not vary more than 10 per cent. from what was computed.

Mace Moulton, M. Am. Soc. C. E.—I will preface my closing remarks by stating that I was not called in to assume charge of the superstructure (as may be noted on page 120), nor were any details submitted for checking until a year after the closing of the contract, based on the specification and general plans, and was not consulted regarding either specifications, plans or contract; and whatever changes in specifications were made subsequently were necessarily such as should not materially increase the cost to the contractors. Whatever opinions I may have held as to the vital features of the specifications or design were not solicited or offered, and consequently my duty was to see that the best structure possible to get under the existing contract was obtained.

The result is certainly far from being a discredit to either the designers or the builders, and may properly be considered a first-class structure. Mr. Schaub deplores the fact that no discussion was offered regarding the general design. The skeletons of original and final designs give data for any one interested enough in the matter to go into it to see the relative economy of the two.

The specifications, in using Launhardt's formula, certainly allow high stresses in some members with small proportion of live load, but also provide well for members first affected by the live load by adding percentages.

I do not feel called upon to defend the use of Launhardt's formula either with his values for "u" or those given in the specifications, and I have never designed a structure under it unless so instructed by the

purchaser. I think, however, that the prevalent deprecation of the great labor involved in its use is uncalled for. We are certainly nearing the time when all will admit that we should treat the live and dead loads separately, either to different factors of "ignorance," as Mr. Cooper calls them, or to some other application, to cover the effects of impact and repetitions of stress, and having the stresses calculated separately it is easy to place them in whatever relation seems most exact.

I studied the record of Wohler's and Spanenburg's experiments carefully some years since, and did not become a convert to the formula as Launhardt made it either in his value of "u" or in the co-efficient in the denominator. Regarding the loads used in the specifications which Mr. Wilson considers too low, I would say that in getting together the test train, I was able, by waiting two or three days, to get the engines corresponding to the diagram given, but no railroad running into Louisville could furnish a train averaging 2 240 pounds per track foot.

The projectors of the enterprise needed a bridge badly, and they built what they considered would answer their requirements and at the same time come within the appropriation. Mr. Schaub objects to the use of the close driven pin at the intersections of the diagonals, and makes some statements regarding the condition of compression members at points so pinned that I am unable to discuss for want of actual knowledge regarding the true state of affairs. I regret that he has not presented his new facts to accompany his assertions so that we might learn exactly how the case stands. In construction, I was assured that the intersecting pins were so located that the compression members should be least distorted when spans were fully loaded, *i. e.*, when camber was nearly taken out and members under nearly maximum stress. I had no opportunity to go into the calculation at the time, but have great confidence in the accuracy of the engineer who adjusted the matter. The contract was based on design with intersection pins, and the members were calculated on the basis that the post was held at the pin and as flat-ended at that point with a length equal to the distances between pin centers.

The tests on posts were made long after every feature of design was settled, and were in no sense used to determine points in the design, but were made for the purpose of ascertaining what they would stand and how they would act in the testing machine. The test on compression steel post proved by its slight set after stresses sufficient to distort rivets in splice that we are getting satisfactory compression joints from the rotary planer. I don't remember seeing a record of a spliced test post heretofore.

I do not agree with Mr. Schaub in wishing we may ever get to the point of ceasing to ream or drill rivet holes. The injury to the hole from punching is certainly there, even in iron, as experiments have plainly shown; and in steel, of whatever grade, the injuries will, in my opinion, seriously impair the strength of the piece, owing to the ten-

dency of every crack, though infinitesimal, to start a fracture. I have often noted the fact that pieces subjected to breaking stress would start a facture at some slight scratch or punch mark, and in some cases the fracture would occur at a point where its sectional area exceeded that of some other place in the piece.

Regarding the planing the edges of plates: The specifications called for universal mill plates, and there was no case in the construction of the entire structure where any other plate was put in to take direct stresses. In case of shearing off corners in compression members, such as in ends of posts or ribs of shoes, both beyond the pin, I did not ask them to be planed. In cases of links made of plate-steel or any other small pieces, they were, if sheared or punched out, put into the annealing furnace and annealed with the eye-bars.

I do not remember a single case where it was considered necessary to plane the edge of a plate on account of its having been sheared. The tests in sheared plates were made for the purpose of adding to our stock of information on the subject. The points which Mr. Wilson brings out as not being mentioned in specifications were all properly attended to in the working up of the designs. The specifications were so long that only extracts were given in the paper, several points which are common to almost every good specification being omitted. The lateral stresses were calculated under a live load, but its effect was smaller than usual, as the train must be nearly covered by the 8-foot high close fence between track and highway on either side.

I cannot inform Mr. Strobel why the pine plank were placed uppermost, except that it was so settled in the contract. Regarding the connection of floor beams to bottom chords, they are screwed up tight against the chords, and a plate was in every case put on the lower side of the chord, through which and the top splice plate of floor beam the lateral pins passed. The rivets in the connection of these plates to the chords were in all cases proportioned according to the longitudinal component of the lateral stress at that joint.

I am sorry to note a tendency to make light of experiments on full size parts in the the testing machine shown by Mr. Schaub regarding the posts, and Mr. Seaman in case of upset ends. In my opinion the more we test the more we know, and if we cannot get stresses near the elastic limit of the parts in the structure, the next best thing is to put the piece in a machine which will break it. We can certainly get some idea as to what to expect of a member when placed in a structure if we get tests enough of similar members in a testing machine. Now in case of steel lateral bars I took particular pains to have a test made of one of the regular bars, selected at random, because I had no knowledge of upsets on steel bars having been tested before, and I think that this structure was the first in which steel upsets were used.

Great pains were taken in shaping the upset dies to avoid sudden

changes of shape, and the result of the test (although only a single one) was very satisfactory. Roughly cut or poor threads of any kind were not passed by inspectors. As the steel was only used in very large bars (for lateral stresses), the upsets and the threads cut in them were very much in excess of the usual sizes, and their use was amply justified.

Information asked for by Mr. Wagner regarding shop manipulation will be found on page 140. The clevises were cast from crucibles with a head of metal about the same as would be obtained in an iron casting of the same size, and subsequently thoroughly annealed. In response to Mr. Wilson's request for my deductions from tests, I should say that the only point not already touched upon is the important one of how we shall inspect steel with a view to its acceptance or rejection. My judgment is based not only on tests made for this structure, but on observations in and about the mills, and of other structures under contract at the same time and later, the specifications for which differed materially from those of this bridge.

1st. Settle on what is desired as to the physical qualities of the steel for the purpose for which it is to be used.

2d. Buy material with those physical qualities, the same to be determined by tests from the finished material, taking not crop ends, but good material for the tests. Steel manufacturers can produce structural steel of any desired grade, and, as they know more than any one else about the amount and effect of work on the steel, they must be allowed to make their steel as they like, provided they produce the desired result. That they can get results to suit the purchaser was forcibly impressed on me on being shown tests two years ago under a specification for a very large quantity of varied material which required that the finished plates and angles should have the same physical properties as the ¾-inch round ingot test-pieces within narrow limits. The manufacturers did it, and that, too, with Bessemer steel; and if they could do it then they can do it again, and hence I think a buyer may with justice expect to get what he is paying for and not be asked to accept something which simply was once like what he demands, but which now, owing to variations in work put upon it, may vary 10 or 15 per cent. from what is designed to use in the structure. The lower the grade of steel called for, the more certain are the results and the fewer rejections liable to occur, and hence the cheaper for the mill and, of course, for the purchaser. Except in large structures the difference in favor of quite high steel is small, and there is where it seems reasonable to exercise the greatest caution in the use of the high grades.

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(Vol. XVII.—October, 1887.)

# TRIPLE THERMIC MOTOR: DESCRIPTION, OPERATION AND RESULTS OF A SINGLE EXPANSION, NON-CONDENSING STEAM ENGINE, SUPPLEMENTED BY THE EVAPORATION OF THE BISULPHIDE OF CARBON AND EXPANSION OF ITS VAPOR, AT BRUSH ELECTRIC LIGHT COMPANY, CLEVELAND, OHIO.

BY CHARLES H. HASWELL, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION JULY 1ST, 1887.

## CONSTRUCTION.

*First.*—An ordinary horizontal and cylindrical fire tubular boiler.

*Second.*—A tubular generator in form of a cylinder boiler set horizontal in which the material of vaporization, known as bisulphide of carbon (formula $CS_2$), is vaporized, having attached in the vapor space an ordinary perforated dry pipe.

All of which is inclosed in a shell having a diaphragm plate between the outer and inner shells at both sides and at one end, thus forming an upper and lower chamber around it. The opposite end is inclosed with a deep disc or bonnet, thus forming a communication between the lower and upper series of tubes, for the proper circulation of the steam with which the $CS_2$ is vaporized.

*Third.*—An ordinary horizontal non-condensing jacketed steam engine.

*Fourth.*—Conduit or vapor-pipe, steam jacketed by being concentric

with one extending from the generator to the cylinder of the engine, the jacket of the conduit communicating with the jacket of the cylinder, and from thence the condensed steam is led by a pipe to a steam trap communicating with the feed pump of the boiler.

*Fifth.*—An automatic regulator or pressure reducing valve, for controlling the admission of steam to the shell surrounding the generator; operated by the pressure assigned to the generator; thus holding the vapor pressure uniform, by admitting more or less steam to the shell as the variation of the load on the engine may require.

*Sixth.*—An automatic reducing valve for controlling the pressure in the jacket around the vapor conduit and cylinder.

*Seventh.*—A heater having within it a coil pipe, through which the condensed $CS_2$ is forced back into the generator.

*Eighth.*—An ordinary surface condenser.

*Ninth*—Three ordinary and small independent steam pumps, and a connection to a water main or centrifugal pump, from which the water of condensation is obtained.

The generator, *i. e.*, the vessel in which the material of vaporization $C\,S_2$ is vaporized, and by which the expansive force is obtained wherewith to operate an engine for motive power, is charged with it to a little over one-half its capacity.

Steam, previously generated in a boiler or primary motor, is led by a pipe to and through the automatic regulating valve, where it is reduced in pressure and consequent temperature; thence to the generator through a perforated pipe between the shells below; thence flowing around the lower half of the generator shell; thence through the lower series of tubes; thence through the upper series; and thence between the shells above; thus circulating through, and radiating heat to the entire surface of the generator: $CS_2$ taking up the latent heat and a portion of the sensible, thus the steam is condensed and gravitates to the bottom of the outer shell; from thence to the boiler feed pump (being one of the three referred to), with the condensation from the jacket of the conduit and cylinder, as delivered through the steam trap; and from thence returned to the boiler.

Steam is also admitted through the reducing or regulating valve, to and through the jacket of the conduit to the jacket of the cylinder, where it is restricted to a reduced pressure, and as it is at a temperature due to this pressure, it is at a temperature in excess of that surrounding the generator, thus imparting an increased temperature to the vapor and effectively superheating it in its course to and in the cylinder of the engine.

The vapor generated from $CS_2$ in the manner described, is delivered through the dry pipe to and through the conduit to the cylinder of the engine, and from the greater temperature surrounding the conduit and cylinder, the vapor is increased in temperature and consequent volume from its admission into the conduit and cylinder, until it reaches the point of cutting off, after which it continues to take up the surrounding heat, thereby supporting its expansive force until it has completed its function in the cylinder.

The exhaust vapor from the cylinder passes around a coiled tube in the heater referred to; thence to and through a surface condenser, from which it is drawn off by the second of the three pumps and delivered into an auxiliary condenser (through the tubes of which the circulating water is first introduced from the main; from thence through the tubes of the condenser; and thence discharged, as in a sewer), thus attaining perfect condensation of the vapor.

Any air drawn from the condenser by the air pump passes through a vessel termed the washer (which is partially filled with water) and thence to a waste pipe.

The liquid $CS_2$ gravitates from the auxiliary condenser to a reservoir. From thence it is drawn by the third of the three pumps and delivered through the coil in the heater (where it absorbs heat from the exhaust vapor on its passage to the condenser); thence to the generator, where it is again vaporized.

An entire plant designed for the development of the practicability and economy of this design of engine has lately been constructed for the Brush Electric Light Company, at Cleveland, Ohio, and on the 31st of May a test of it was made under the direction of Mr. Isaac V. Holmes, representing the Light Company.

### Elements of the Plant.

An ordinary horizontal fire tubular boiler set in masonry having a grate surface of 16.5 square feet and a shell and tube surface of 225 square feet.

Combustion, natural draught. Coal, anthracite.

An ordinary jacketed non-condensing engine, single expansion, having a diameter of cylinder of 14 inches and a stroke of piston of 36 inches.

Generator, having a diameter of 54 inches, a length of 15.25 feet and a steam heating surface of 1 550 square feet.

Surface condenser having tube surface of 1 000 square feet.

The operation of the engine was contined without interruption for a period of five hours, which, inasmuch as that period involved the cleaning of the fire in the furnace of the boiler, was held to afford full time for a test of the operation and the effects of the elements under investigation.

RESULTS.

| | |
|---|---|
| Pressure, steam—Boiler | 75.8 pounds. |
| " " Shell | 15.3 " |
| " vapor—Engine | 76 " |
| " " Mean, by indicator | 31.35 " |
| Water evaporated | 5.71 cubic feet. |
| Revolutions per minute | 100 |
| Vacuum | 9.85 pounds. |
| Coal consumed | 600 " |
| Horse power indicated | 86.64. |

NOTE.—Pressures are given in pounds mercurial gauge, and temperatures in degrees of Fahrenheit.

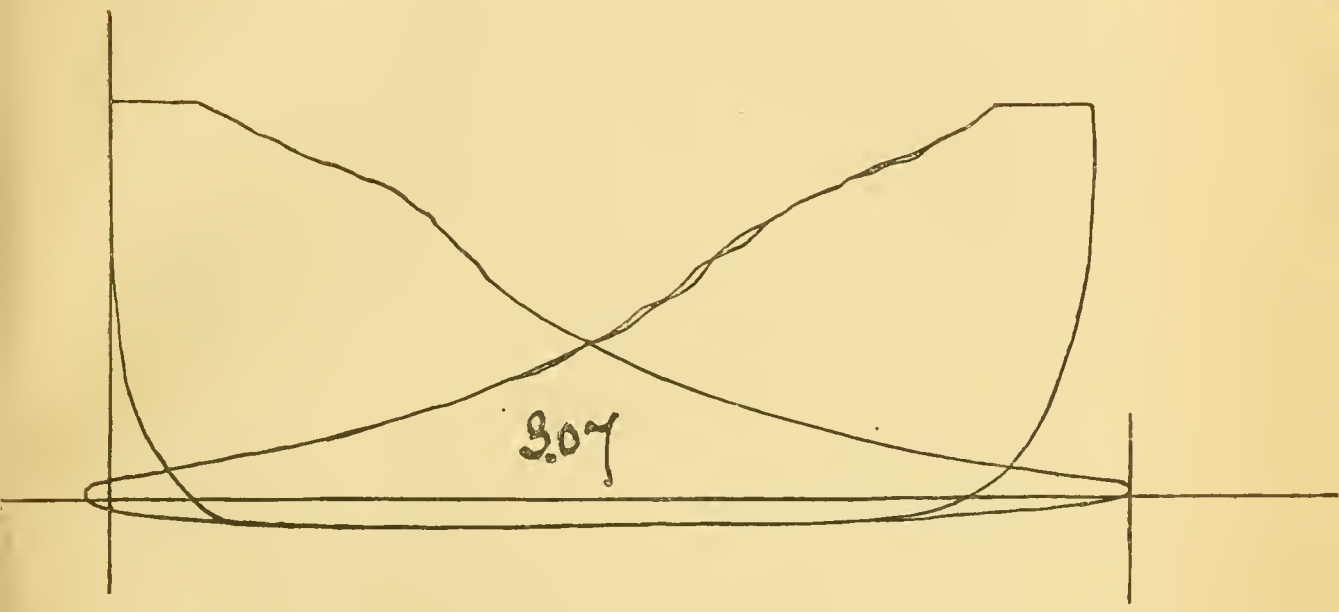
From which it appears that steam at a pressure of 75.8 pounds per square inch, passed through the automatic regulating valve to the shell surrounding the generator at the reduced pressure ef 15.3 pounds, due to a temperature of 250.4 degrees, produced a vapor in the generator of 76 pounds.

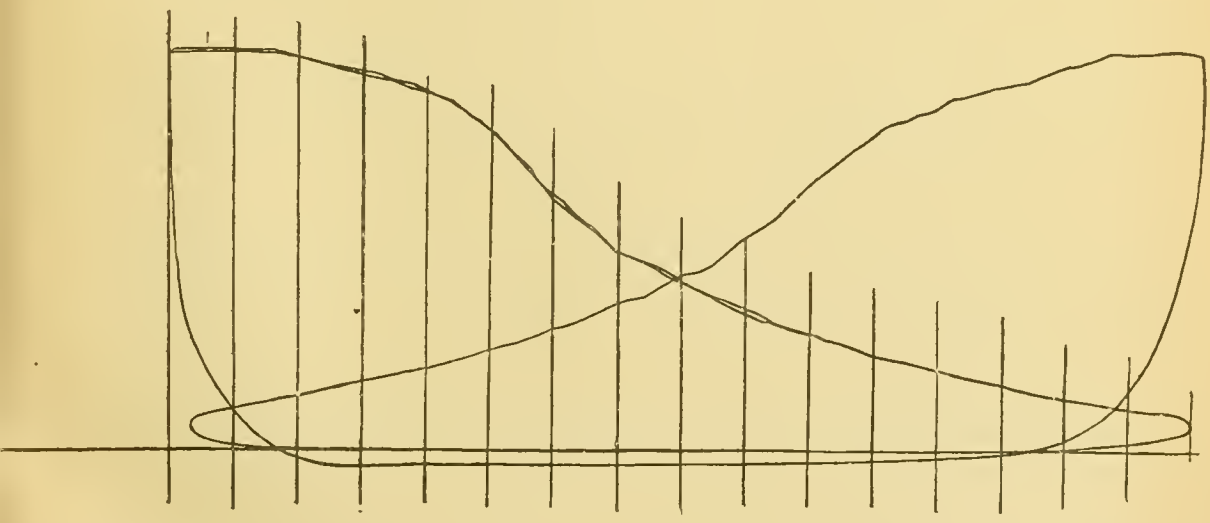
| | |
|---|---|
| Sum of sensible and latent heat of steam at one atmosphere | 1 178.1 degrees. |
| Sum of sensible and latent heat of $CS_2$ | 274.4 " |
| Latent heat of steam at one atmosphere | 965.2 " |
| Latent heat of vapor of $CS_2$ at one atmosphere | 156.4 " |
| Sum of sensible and latent heat of steam at 45 pounds (the pressure of the steam surrounding the vapor in the conduit) | 1 206.6 " |
| Sum of sensible and latent heat of the exhaust steam at one atmosphere, as defined by the indicator cards | 1 178.1 " |

Hence, 1 206.6 minus 1 178.1 equals 28.5 degrees expended in the cylinder.

| | |
|---|---|
| Sum of sensible and latent heat of $CS_2$ vapor at 76 pounds pressure, and superheated by steam at a temperature of 292.3 degrees (45 pounds) is | 443 degrees. |
| Sum of sensible and latent heat of $CS_2$ at one atmosphere, that being the temperature of the exhaust, is | 274.4 " |

AREA, 3.07



CARD ILLUSTRATIVE OF EXPANSION OF THE VAPOR.

PLATE XXII.

Hence, 443 minus 274.4 equals 168.6 degrees expended in the cylinder, and 168.6 divided by 28.5 equals 5.916.

Therefore, the relative theoretical value of the vapor of $CS_2$ as compared with that of steam is as 5.916 to 1; or, in other words, the heat admitted to the shell around the generator, additionally heated by the excess of that of the steam around the conduit and cylinder, will produce an elastic vapor in the cylinder of an engine 5.916 times greater than if steam alone was used.

Or, steam at 15.3 pounds pressure generates a vapor of 76 pounds pressure, which when additionally superheated by being enveloped in steam at 45 pounds, its volume is increased and its pressure of 76 pounds is fully maintained at the cylinder of the engine.

In the test, the elements of which are here recorded, the consumption of coal per hour was 120 pounds.

Indicated horse power, 86.64.

Hence 120 divided by 86.64 equals 1.385 pounds per indicated horse power per hour.

The volume of water at 50 degrees (the ordinary or mean temperature of condensing water) required to condense steam at the temperature of that of the vapor which passed through the heater (212 degrees, as determined by the indicator card) and deliver it to the boiler at the ordinary temperature of 100 degrees, is as 22.95 to 1, the volume evaporated.

The volume under like conditions for the condensation of the vapor is but as 3.67 to 1, to the volume evaporated.

Hence, when $CS_2$ is compared with steam, a less area of condensing surface and less volume of condensing water is required, both of which conditions involve an economy in the cost and endurance of an engine.

Reviewing then the elements submitted, it is presented, that by the use of the vapor of bisulphide of carbon in a vessel connected to an ordinary steam boiler and engine, condensing or non-condensing, an elastic vapor can be attained greatly in excess both in pressure and volume of that of the steam that generated it; and, as an evident and infallible consequence, both increased power and economy of fuel are attained with less boiler surface and consequent wear.

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# THE DAVIS (CREVASSE) LEVEE.

By SIDNEY F. LEWIS, M. Am. Soc. C. E.

READ JULY 7TH, 1886.

On March 8th, 1884, at about one o'clock in the morning, a break occurred in the Davis Levee, situated on the west bank of the Mississippi River, some twenty-three miles above the City of New Orleans.

The crevasse, in the official report of the Board of State Engineers for the years 1882–83, and to April 20th, 1884, is said to have been caused by the washing out of loose earth in an imperfectly refilled rice flume cut in the levee. The levee at this point was upwards of eight feet high, and the water in the river on that date was within eighteen inches of the top of the levee. The gauge at New Orleans* read one foot one inch below high water mark of 1874.

The position of the break was near midway in a bend of the river, where the current impinges directly against the bank. The width of foreshore about sixty feet, with a depth abrupt to it of over fifty-eight feet at a low stage of the river. This sixty feet of foreshore was cut away in a short time.

* The zero of river gauge at New Orleans was changed on November 1st, 1885, from high water of 1874 to low water of 1876. This change makes the readings 16.2 feet lower than those made previous to November 1st, 1885.

On March 27th the dimensions of the crevasse was found to be as follows:

| | | |
|---|---|---|
| River below flood stage..... | 0.5 | feet. |
| Width of outlet..... | 557 | " |
| Area of outlet..... | 11 640 | square feet. |
| Reduced mean velocity per second ..... | 12.9 | feet. |
| Discharge through outlet per second..... | 140 156 | cubic feet. |

After subsidence of high water it was found to be 1 400 feet wide.

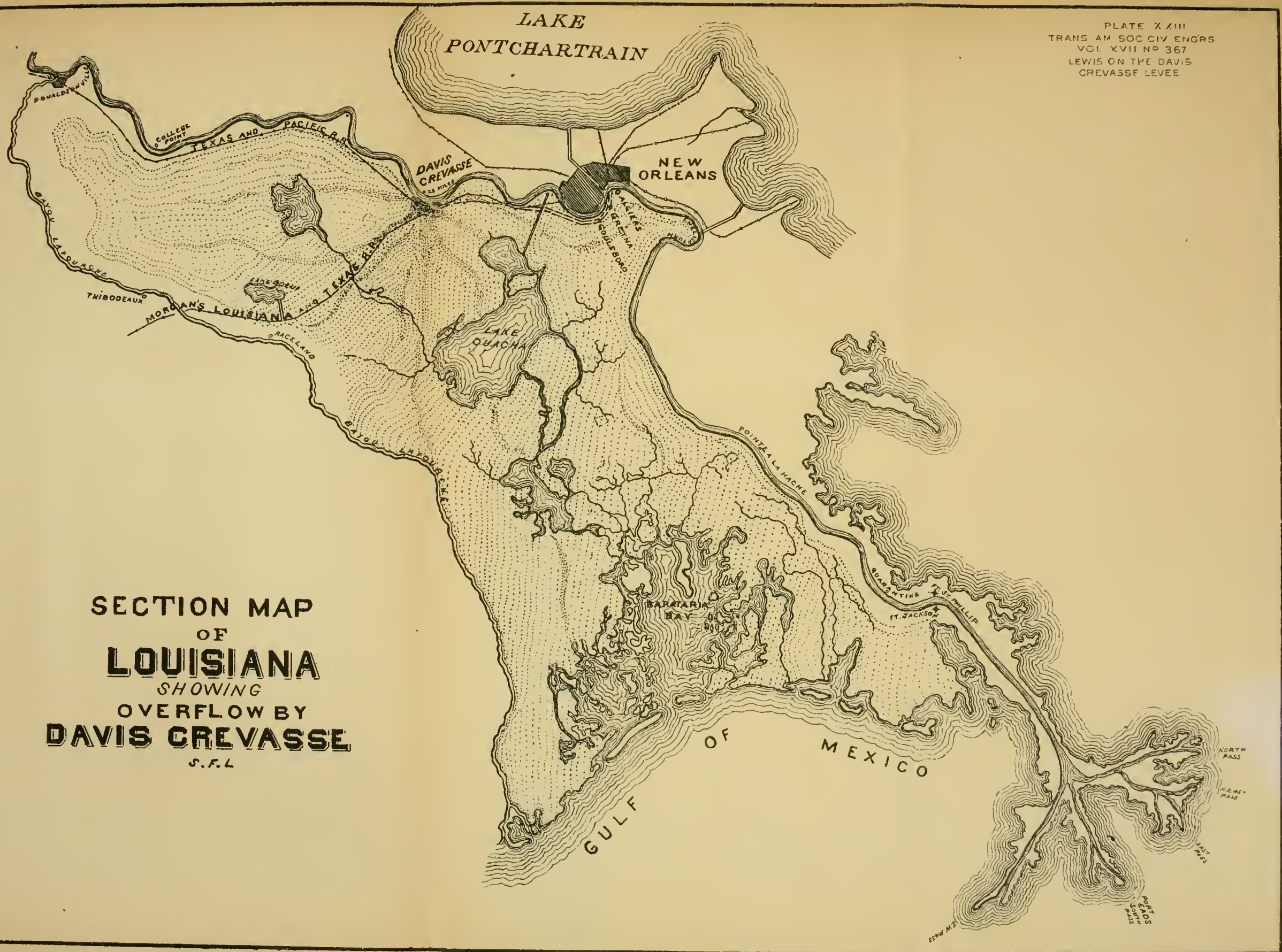
In the incipient stage of the break an attempt was made to close it by the Texas and Pacific Railroad Company and Morgan's Louisiana and Texas Railroad Company, their tracks being submerged by the waters of the crevasse, but after considerable expense in labor and material their efforts proved futile. In a few weeks a section of country (as represented by the accompanying map, Plate XXIII) some thirty-three hundred square miles in area became an inland sea, separated from the river, as it were, by a green ribbon of levee. Sugar and rice plantations, with the many vegetable and fruit farms, for over sixty miles above the city, and to the limit of planting interests, some sixty-five miles below the city, were covered with water from three to five feet deep. It may be interesting to state that the U. S. steamer Patrol, a stern-wheeler 110 feet long, drawing over four feet of water, whilst in the service of gauging the crevasse, was carried through the break without injury, and was afterwards piloted safely across the overflowed country to Bayou La Fourche, and thence back to the Mississippi River.

The suburbs of New Orleans, Gretna, Gouldsboro and Algiers were mostly under water, with here and there isolated spots, which, by their favored positions or back levees, were able to keep the crevasse water out of the dwellings and yards.

The road-bed of the Morgan's Louisiana and Texas Railroad Company was damaged and submerged for forty-eight miles. The road-bed of the Texas and Pacific Railroad Company was damaged and submerged for over twenty miles. For a period covering more than four months these sections of roads could not be utilized, and passengers and freight had to be transported by boats to their respective termini above overflow.

The following estimated losses of the Morgan's Louisiana and Texas Railroad from Davis crevasse is transmitted:

"The loss in business is based on the decrease in net earnings, amounting to $693 972.48, which result was brought about by:




# SECTION MAP OF LOUISIANA

SHOWING

OVERFLOW BY

# DAVIS CREVASSE

S.F.L

"*First.*—Increased cost of operation by roundabout and expensive routes.

"*Second.*—Diminished revenue.

"*Third.*—The small volume of business done after the reopening of the road, on account of the country tributary to it having been ruined.

"Trains stopped running on March 11th, 1884, and commenced running again on July 26th, 1884.

"It would not be out of proportion to estimate the cost of protecting the forty-eight miles, and of putting them in first-class order after the overflow, at five hundred dollars per mile, or $24 000.

"Money actually spent in attempting to close Davis crevasse, $22 500.

"It is impossible to estimate the losses incurred by the Texas and Pacific Railroad from this crevasse alone. On March 20th the Atchafalaya River overflowed its banks and submerged the track in the swamps, so that the New Orleans Division was not operated south of Bunkie from March 20th until August 15th, when the repairs of road-bed at Davis crevasse were completed, and the line operated through to Gouldsboro.

"Amount expended by Texas and Pacific Railroad in efforts to close the crevasse, including labor and material, about fourteen thousand dollars."

It was not until the latter end of August of that year that the water over the land and in the river had sufficiently declined to make instrumental surveys for a line of levee to inclose the break. As engineer in charge of that levee district, the surveys were intrusted to me. Two lines for a levee were located, one to circumvent the washout or gulch caused by the crevasse, as represented by the dotted line $A\ B^1\ C^1\ D$ (see Plate XXIV), requiring about one hundred and sixty-five thousand cubic yards of earth; and the one finally adopted and built across the channels, as represented by the line $A\ B\ C\ D$, three thousand eight hundred and one feet in length, requiring some one hundred and fifty-three thousand cubic yards of earth.

This line offered considerable difficulties in surveying, as the surface of the ground was very much honeycombed by the waters of the crevasse, and the channels at that stage of the river averaged sixteen feet deep.

On the decline of the river in August, the Texas and Pacific Railroad Company, whose track was nearest the break, had trestled over the channels of the crevasse in pretty much the same line as their original road-bed, and this trestle served the purpose of a base to arrive at an accurate survey and cross-section of the ground adjacent thereto. This line, after due deliberation, was adopted by the Board of State Engineers, not only because it was much the shorter of the two and contained less yardage, but that the levee on this line could the more easily and readily

be built, as the existing trestle of the Texas and Pacific Railroad Company was utilized for the construction of the larger sections of the embankment across the channels, by hauling earth in flat-bottom cars, using a steel plow and cable to unload them, whilst the approaches or wings could be built with wheelbarrows, and the construction of these different sections of levee kept in progress independent of each other.

With this purpose the following designs of sections for the embankment were devised and adopted by the Board of State Engineers. (Plate XXV).

*Fig.* 1.—Average cross-section of levee for upper wing or approach.
*Fig.* 2.—Average cross-section of levee for lower wing or approach.
*Fig.* 3.—Cross-section of levee for deepest portion of channel on river side of railroad trestle.
*Fig.* 4.—Cross-section of levee for deepest portion of channel on land side of railroad trestle.

On September 5th, 1884, a force of about one hundred able-bodied men began breaking ground and wheeling earth on the upper wing or approach *C D*.

On September 11th, 1884, a similar force was put to work on the lower wing or approach *A B*. Another camp of about eighty men later on were put to excavating a pit for loading the cars.

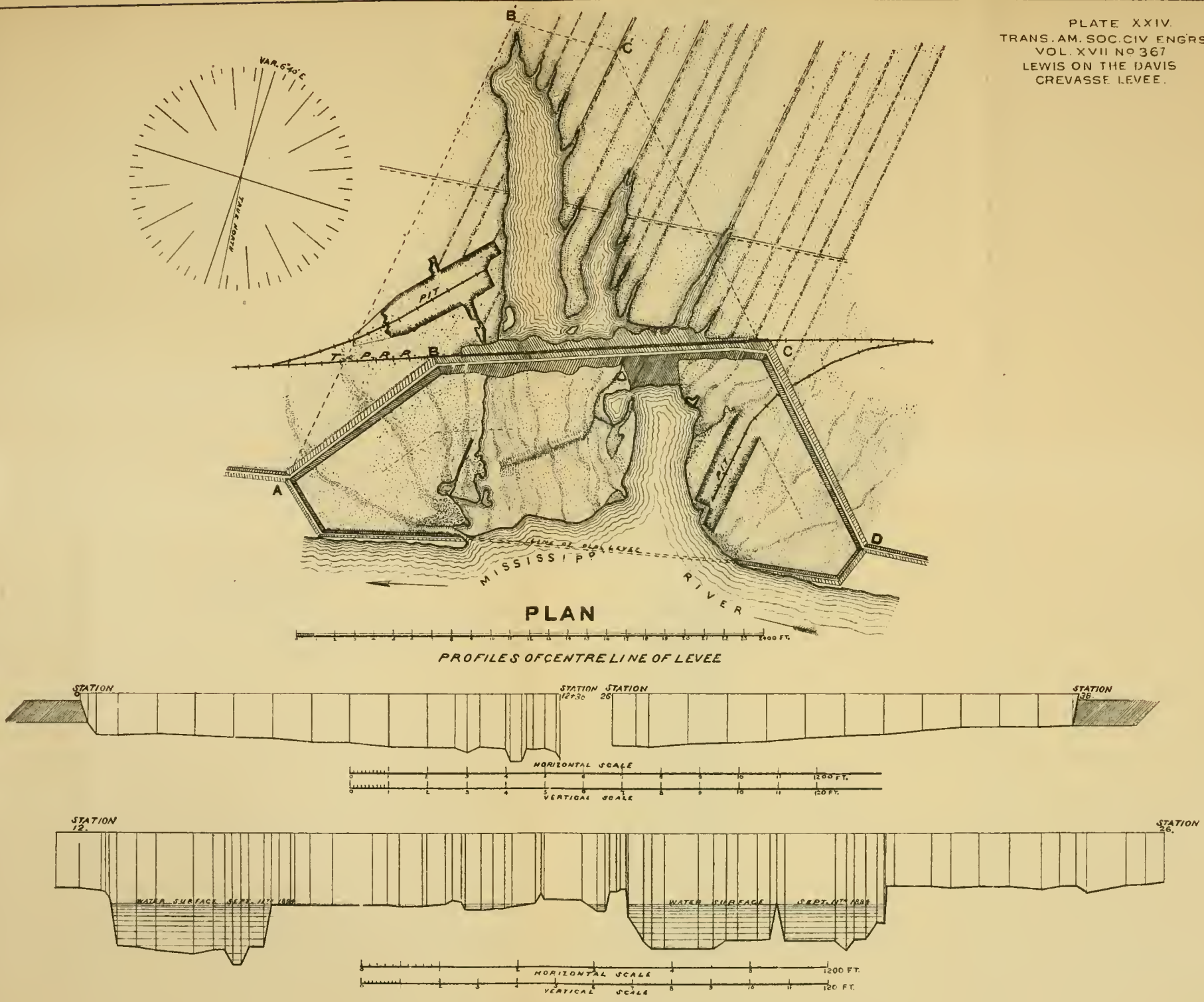
The approaches or wings, aggregating some thirty-eight thousand seven hundred and fifty cubic yards of earth, were completed about October 1st, 1884.

The camps were then concentrated on the curtain or main line of levee, which was partially built with earth hauled in flat-bottom cars, and then topped off with wheelbarrows.

In the months of January and the early portion of February, 1885, the lower Mississippi River attained an unprecedented height for that time of the year, averaging some two feet below the high water mark of 1874 at New Orleans.

During this period some three hundred and fifty feet in length of the embankment across the upper and deepest portion of the channels of the crevasse was in a precarious condition, as scarcely more than one-third of the earth required had been placed in position, and in consequence gave rise to great anxiety to the railroad authorities and other parties interested. The river subsided as quickly as it had risen, and subsequently during the spring floods never attained its February height. This uneasiness led the railroad authorities to drive a row of sawed piles twelve inches square and fifty feet long across the upper channel.

PLATE XXIV.
TRANS. AM. SOC. CIV ENGRS.
VOL. XVII No 367
LEWIS ON THE DAVIS
CREVASSE LEVEE.



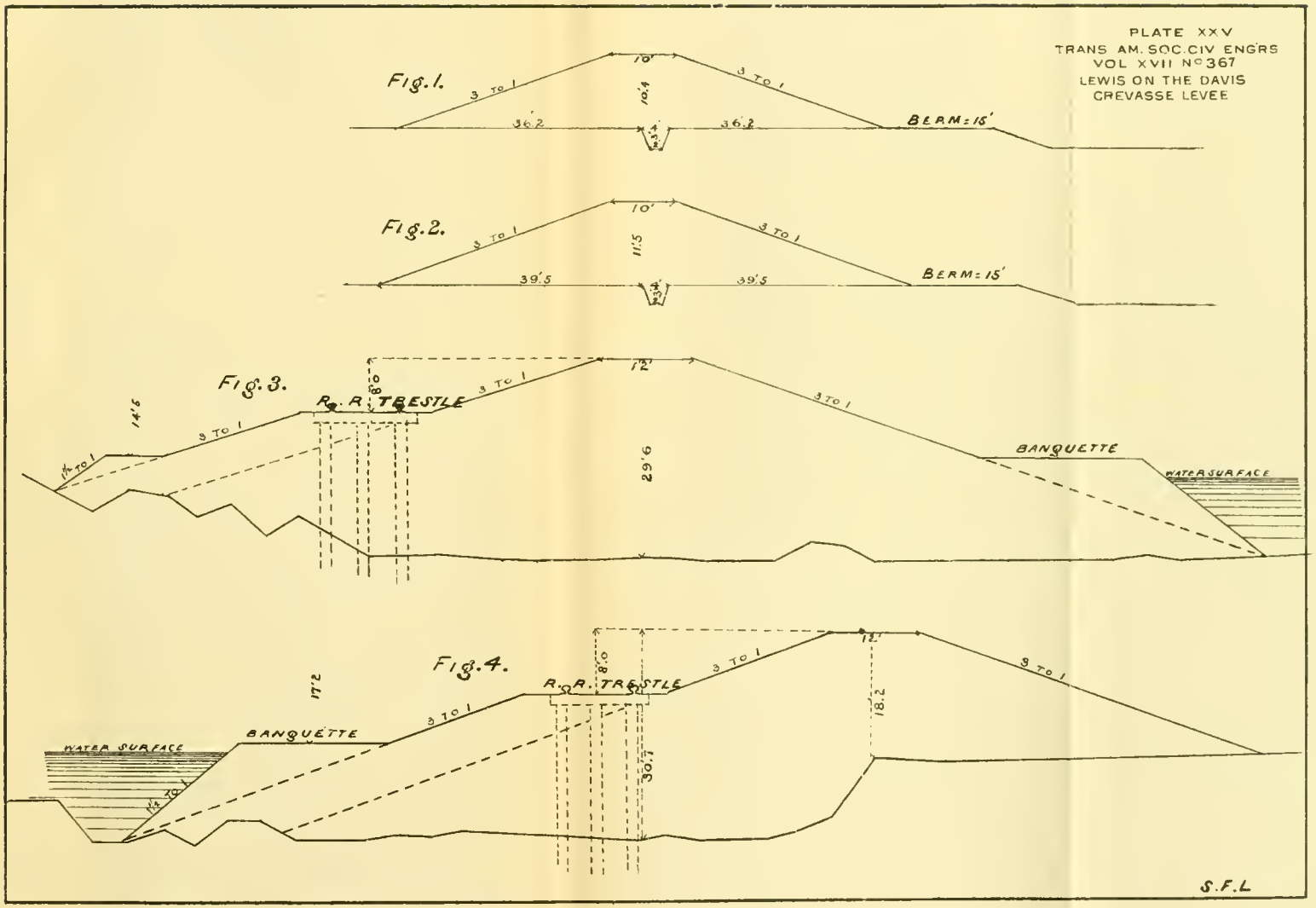

# DAVIS LEVEE

## ST. CHARLES PARISH. RT. BK.

## LA.

S.F.L.

PLATE XXV
TRANS. AM. SOC. CIV. ENG'RS
VOL. XVII No. 367
LEWIS ON THE DAVIS
CREVASSE LEVEE



At the suggestion of the Superintendent and Engineer of the Morgan's Louisiana and Texas Railroad Company, they were driven on the river side forty feet from center of levee when completed, some fifteen feet in the solid ground, six feet from centers, and capped with an horizontal piece of twelve by twelve-inch bolted through the head of the piles.

Sections of the embankment across this upper channel taken on February 9th, 1885, showed the piles to be leaning. As the earth was placed in position these piles continued to give way, until finally they were completely overturned. A second row of round piles outside of these were then driven, which met with a similar fate. The river half of this section of the embankment, as referred to above, would crack and slide with a slow motion towards the river, the tendency and amount of slide increasing or decreasing with the fall and rise of the water, whilst the half section on the land side remained firm and steady with no disposition to change its slope of three to one.

In consequence of this it became necessary to keep a wheeling force of upwards of sixty men during high water season to keep the dump above the surface of the water in the river. When the river had sufficiently declined so as to leave this section of the embankment out of danger, it was deemed advisable to relieve the wheeling force, as it was presumed that the summer months would dry up and help this sliding portion of levee. It was also found that there had been over twelve feet of deposit from the river in the main channel of the crevasse.

In the following fall a force of about twenty wheelbarrows were put to work to complete this section of the embankment, it being estimated that some twenty-six hundred cubic yards of earth would be required to bring it up to grade.

On October 27th the sliding of this section of the embankment began to reshow itself; the wheeling force was kept at work to April, 1886, when it was relieved, as the embankment had about come to its bearings.

The cost of this levee up to date amounts to $38 145. It is probably the cheapest levee for its magnitude ever constructed in this State.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVII.—October, 1887.)

# REPLACING THE STONE TOWERS OF THE NIAGARA RAILWAY SUSPENSION BRIDGE, WITH IRON TOWERS.

By L. L. BUCK, M. Am. Soc. C. E.

READ NOVEMBER 2D, 1887.

It is not the intention in this paper to give a minutely detailed description of the old stone towers of the bridge, but merely such a brief description as will explain their condition, the causes which led to their replacement and the manner in which it was done.

The stone towers rested on the rock which forms the banks of the gorge spanned by the bridge. On the New York side they were about 90 feet in height, from the rock to the center of the cables, where they rest in the saddles. On the Canadian side, the rock being about 10 feet higher than on the New York side, the towers were about 80 feet high. Each tower, of a pair on either side of the river, had a heavy base of rock faced ashlar, extending to about the level of the upper chords of the stiffening trusses of the bridge, and the two bases were connected by an arch to support the railway track, between the towers, and allow of the passage of pedestrians and vehicles to the lower floor. The portion of each tower above the base approximated to the frustrum of a pyramid 15 feet square at the base and 8 feet square at the top. They were built of dressed limestone, laid in Thorold cement mortar. The top of each tower was entirely covered by a cast-iron bed-plate 2½ inches thick. On the upper surface of this bed-plate were cast three ribs, parallel to the direction of the cables, 2½ inches thick by 7½ inches high, and extending across the plate. The two spaces thus left be-

tween the ribs were 2 feet 2 inches wide, and truly planed on their bottoms and sides. In each space were ten cylindrical cast-iron rollers, turned to 5 inches diameter by 2 feet 1¾ inches long, laid close together, side by side, and transversely to the ribs. On each set of rollers rested a saddle of cast-iron, 5 feet long by 2 feet 1¾ inches wide at the base, the under side of which was planed to rest on the rollers. The upper portion of the saddle was provided with the usual U shaped groove in which the cable was laid with an easy curve. Through the upper portion of the saddle, between the base and U shaped groove, were cored nine holes, extending from side to side. These holes are mentioned here for the reason that they served an important purpose in securing the cables to the lifting apparatus.

The stone of which the towers were built was limestone, quarried near the site of the bridge. A few of the first stones laid at the bottoms of the towers were taken from the surface stratum and remained sound; but, for some reason not explained, all the remaining stones were taken from a stratum below, which would not bear exposure to the weather.

Sometime after the completion of the towers the defective character of the stone began to manifest itself. Irregular shaped pieces would become detached and cracks appeared in various parts of the surface. Painting was resorted to as a preservative, and was continued at intervals, as long as the towers were retained.

My first experience with the bridge was in 1877. At that time the surfaces of the towers were considerably disintegrated, and pieces had fallen off here and there. Most of the cracks were in every conceivable direction, but there were a number of cracks which took a nearly vertical direction, generally indicative of pressure being their cause.

But little attention was given to the towers at the time of their inspection that year, for the reason that the condition of other portions of the bridge was apparently of more importance.

New face stones had from time to time been inserted. But in 1880, while the work of rebuilding the suspended superstructure was in progress, the surfaces of the towers were in bad condition, numerous pretty large pieces having fallen off, and a good many more were loosened and ready to fall. At that time I noticed that, in external appearance, the most defective portions of the towers were those located about one-third the distance from the level of the railway floor to the top, and that the disintegration gradually decreased as the distance above and below this

point increased. This appearance could only be accounted for by supposing that the towers were being subjected to a considerable bending stress, which by opening the joints, cracking the paint and admitting water, coupled with the increased pressure at the surface, and the bad character of the stone, would fully account for it. Moreover this was the part of the tower subjected to the greatest bending stress.

The cause of this bending stress was the elongation and contraction of the cables between the towers and the anchorage, produced by live load stress and changes of temperature. This portion of the cable is as nearly straight as its own weight alone will allow the tensive stress to which it is subjected to make it; the deflection averaging about 15 inches. The maximum movement at the top of the tower, produced by the live load, is about $\frac{5}{8}$ inch, while that produced by temperature, between extremes, is a little over two inches. As these two act in concert only at the higher temperature, it is evident that the greatest movement* of the saddle at extreme high temperature would be about $1\frac{5}{8}$ inches from its normal position and toward the river, while in the opposite direction it would move from the normal position to that at extreme low temperature, about 1 inch.

In 1880 the greatest movement of the saddles with respect to the bed-plate was $\frac{1}{8}$ inch. The rollers were so situated as to be inaccessible excepting by cutting away the outer ribs of the bed-plate, but these ribs were the only guides to prevent the saddles from working laterally, and as events have since proved, it could not have been of any benefit.

After completing the suspended superstructure of the bridge, I was requested by the Board of Direction to report upon the condition of the towers and as to what ought to be done upon them.

My examinations had already convinced me that any plan for buttressing the towers or otherwise attempting to prevent their bending would prove abortive. If encased in sheet iron nothing could be known of the changes taking place within, and as the motion would still continue, it was very doubtful whether such covering could be kept water-proof.

The only plan that promised any success appeared to be that of cutting away the defective portions of the stone to various depths or till the sounder portions within were reached, and then reface the towers

* Since the above was written the movement for the present winter between + 40 degrees and zero has been $1\frac{1}{2}$ inches.

with new stones, thoroughly anchored to the interior portion, and cement and grout all interstices.

There appeared to be a possibility that if this work was well and carefully done, the new covering would possess sufficient flexibility to allow of the swaying of the tower without crushing the new stone. At that time it was supposed that the interior portions of the towers were sound.

My report favored this plan as an experiment, and it was decided to have the work done. The refacing was completed in the latter part of the year 1883; the last tower treated being the north one on the Canadian side.

During the summer and fall of that year I was in Portland, Oregon, and consequently saw very little of the interior portion of this last tower, as disclosed by the cutting away. On my return from the West I was informed by Mr. Patten, who had charge of the work, that this last one had two cracks in it of such a serious nature that they gave him considerable uneasiness. One of these cracks started at the middle of the top of the south side, and the other at the middle of the top of the west side. Both extended downward, gradually approaching the southwest corner, where they met at a point above the railway floor about one-third the height of the tower above that floor.

A close watch was kept upon the towers to detect any change that might occur.

In September, 1885, I inspected the towers. At that time the north tower on the New York side was apparently in fair condition. The south tower, on the same side, had opened some of its joints slightly, and two or three of the new facing stones were slightly cracked. On the Canadian side the south tower had opened some joints and cracked some of the new stones. A serious crack had also started in the old stones on the east face, beginning about twelve inches from the northeast corner. Thence it extended upward and toward the middle of the face, through three courses. This was more serious than the cracks in the new facing, for the reason that as the old stones were originally very large, such a crack must necessarily extend to a great depth.

The north tower was in a far worse condition than any of the others. Many of the joints in the new faces had opened; but, on the south and west faces, the joints had not only opened but many of the stones were badly cracked, and the southwest corner, at the point above mentioned, appeared about ready to fall out.

At this time, motion of the saddles on the rollers had about ceased. This could have been remedied by lifting the saddles, removing the old rollers, and by using a larger number of rollers of less diameter with new planed plates above and below them, and then by refacing the towers, they might be made to stand some time longer. But the expense for apparatus and refacing would be considerable, while the internal condition of the towers would still be uncertain. Moreover, all the rollers on top of one tower only covered an area of 3 feet 4 inches by 4 feet 7 inches equaling 17.18 square feet. The bed-plate could not effectually increase the area of distribution of the weight to more than 25 square feet. The maximum weight of about 900 tons resting on the rollers would cause a pressure of 36 tons per square foot on the masonry. As shown by sounding the plates around the edges with a hammer, the area given above as actually being pressed upon is certainly not understated. There was consequently a probability that (considering the character of the stone in the towers) the interior stones at the tops of the towers would be found unsound.

These considerations decided me to report in favor of replacing the towers with iron.

The following winter I prepared plans for the new towers, and the Board of Direction having decided to act upon the recommendation, the contract for the iron work was let to the Detroit Bridge and Iron Works.

The plans for the new towers, method of erection and of transferring the cables were according to the following description (see Plate XXVI).

Each iron tower is made up of four wrought-iron columns, braced transversely and longitudinally by wrought-iron struts and rods. Each column rests upon a limestone pedestal, founded on the rock and capped with granite. The outer portions of the old bases were cut away to make room for the new pedestals. The tops of each pair of columns are connected by a heavy web plate secured to each web of each column by angles and rivets. Their tops are covered by a wrought-iron plate truly planed to present a level surface when in position, thus forming a cap. On the caps of the two pairs of columns rests the main bed, built up of wrought-iron plates and angles. It is in plan 9 feet 2 inches long by 5 feet 3 inches wide and in height 3 feet 8 inches. Its weight is about 9½ tons. The whole of its top surface is truly planed. On it are laid flat, with no fastening, two steel plates, each 2 feet 2 inches wide, ⅞ inch thick by 7 feet long, with both sides and both edges planed

parallel. These plates are laid with their long sides parallel with the direction of the cable. On top of the steel plate, and laid transversely, are eighteen steel rollers, turned truly cylindrical, with diameters of 4 inches and length between shoulders equal to the width of the steel plate. The ends of each roller have trunions which enter into holes in the side pieces of the roller frame. The side pieces of this frame are 1 inch thick by scant 5¾ inches wide, and have their inner faces and both edges planed so as to come into neat contact with the edges of the steel plates and shoulders of the rollers. Resting on the rollers is a cast-iron bed whose base is 7 feet long by 2 feet 4 inches wide, height 18 inches, and top 5 feet by 2 feet 2 inches. The top and bottom surfaces are planed parallel, and to the under surface is riveted a planed steel plate of the same dimensions as that under the rollers. By this arrangement the rollers are protected from water and, as far as possible, from dust. On this casting rests the old saddle. In addition to the bracing of each tower, each pair of towers is braced together transversely.

The transferring apparatus consisted of four cast-iron columns, two transverse and two longitudinal girders, six 125-ton hydraulic jacks, a quantity of wrought-iron shim plates and of fine quality of No. 8 cast-steel wire.

When the columns of the towers had been erected in place, the transverse bracing was put in immediately, but the longitudinal bracing could not be put in till the stone towers were removed. Consequently their place was temporarily supplied by clamping the iron work to the stone towers.

After the columns had been so placed and clamped, the temporary cast-iron columns were set up, one at each end of each cap, when their upper ends extended above the cable, and the space between a pair of the cast columns was such as to allow of passing the large bed between them. On top of each pair of cast-iron columns rested a transverse girder, and on the two tranverse girders rested the ends of the two lifting girders. When in position, each lifting girder was directly over and parallel to a saddle. The saddles were then secured to the lifting girders by passing the steel wire through the before-mentioned holes in the saddles and up over cast-iron blocks on the lifting girders. The wire was thus passed continuously around with three hundred and fifty turns for each saddle.

The six jacks were then set under the transverse girders and resting

on the caps of the wrought-iron columns, in the space through which the bed was to pass—three jacks under each transverse girder. The bed was then hoisted on the side toward the river and temporarily supported, close under the cables, by a wrought-iron frame, the end of the bed next to the tower resting on rollers to allow of its being easily moved. The cast-iron saddle beds, rollers and steel plates were then raised and conveniently placed on the staging, between the towers.

The six jacks were then operated simultaneously, thus lifting the transverse girders, which raised the longitudinal girders, carrying the saddles and cables with them. As the transverse girders raised, the iron shim plates were inserted, between their ends and the tops of the cast-iron columns, till a sufficient number of the plates had been inserted to keep the saddles clear of the old rollers, when the jacks had been discharged. As this required a considerable number of the plates, it was necessary to drive them in pretty firmly with a hammer, otherwise their irregularities would keep them sufficiently separated to allow the heavy weight to fall through a considerable distance and produce a disagreeable shock. After the whole weight had been thus tranferred to the cast-iron columns, the jacks were again forced up against the girders, with a fair pressure, for additional safety. The weight now rested on the iron work.

The old bed was next raised, clear of the masonry, and suspended by rods to the lifting girders. The upper three courses of masonry were then removed. The jacks were then set to one side, and into the clear space, thus made for it, the large wrought-iron bed was moved, correctly placed, and bolted to the caps. The old bed was then lowered onto the new one, the rollers taken out and thrown to the ground, and the old bed-plate temporarily placed on the iron frame which had supported the new bed.

It now remained only to place the steel plates, rollers and saddle beds on the new bed, set the jacks on the ends of the new bed, underneath the transverse girders, lift them high enough to release the shims, which were then removed, and by simultaneously discharging the jacks, the saddles were allowed to settle permanently onto their new beds. This completed the transfer for that tower. Each of the four towers were successively treated in the same manner.

The stone towers had then to be removed. While this was being done it was necessary to temporarily supply the place of the longi-

tudinal bracing, by wooden shores against various parts of the masonry. As rapidly as the sections of iron work were uncovered, the iron struts and rods were put in and secured.

After all the preparations for a transfer were completed, the time consumed in making the change, including the removal of the top three courses of masonry and the old bed-plate, was eight and one-half hours, excepting in the case of the first one which required longer time.

Trains were not allowed on the bridge while the transfer was being made.

The total weight raised by the six jacks was about 650 tons. Two men were at each jack to operate it.

### Condition of the Old Towers as Developed by the Progress of the Work.

In cutting away the faces of the old towers at the tops, the stones were found to be cracked through vertically, some of them in several places. Some of the stones were not more than half bedded near the top, so that, in some cases, a two-foot rule could be inserted its whole length and swung around in the space. In such cases the space had to be filled with cement before the cutting was allowed to proceed.

When the weight was lifted from the old bed-plate, it raised in the middle nearly a quarter of an inch. In removing the top three courses, the interior stones were found to be badly shattered in every direction, and in barring them out they flaked into spawls. While the stones were thus broken up, the cement gave indications of having been excellent.

When access to the old rollers was obtained they were found to be embedded in cement and iron rust, the spaces between them and between their ends and the ribs of the bed-plate being filled with this cement, which had become so firm that a chisel bar was required in extricating the rollers. Thus each roller was lying in a trough, and whatever water found its way in there could only escape by evaporation. While this cement had originally allowed the roller some motion, the corrosion caused by the water operating upon the iron surface, kept clean by the heavy rolling pressure, had flattened the bottom sides of the rollers more than $\frac{1}{32}$ inch, and produced depressions in the bed plate to the depth of $\frac{3}{32}$ inch, and in these depressions, assisted by the iron rust and cement, the rollers had come to have a very snug bed. The top of the

roller and under side of the saddle, although slightly rusty, were all right and true.

In view of the condition of the top courses of masonry and of the rollers, there is little reason to doubt that had any successful plan been adopted to resist the swaying of the towers, it must have resulted in their prompt destruction by the consequent grinding down of the top.

On the day when the second transfer was made the thermometer stood at + 7°. On lifting the saddles from the rollers the top of the tower suddenly sprung toward the river $\frac{5}{8}$ inch. It can only be conjectured as to how much stress was imposed upon the anchorage in producing this bending of the tower. But it must have been considerable. One circumstance which occurred will aid such conjecture to some extent. The tops of the towers on the Canadian side were found to be about two inches nearer together than their bases, and an effort was made to force them apart with one of the large jacks. The distance was increased $\frac{1}{4}$ inch with little trouble, but when it had increased $\frac{3}{8}$ inch the timber struts, against which the jack was pressing, began to buckle and the attempt was abandoned. There was at that time a probable pressure of 60 tons on the jack. When it is remembered that the $\frac{3}{8}$ inch is the amount which 60 tons increased the distance by bending the two towers in opposite directions it will be seen that bending one alone $\frac{5}{8}$ inch would require a very much greater force.

The removal of the old towers was a somewhat tedious process for the following reasons:

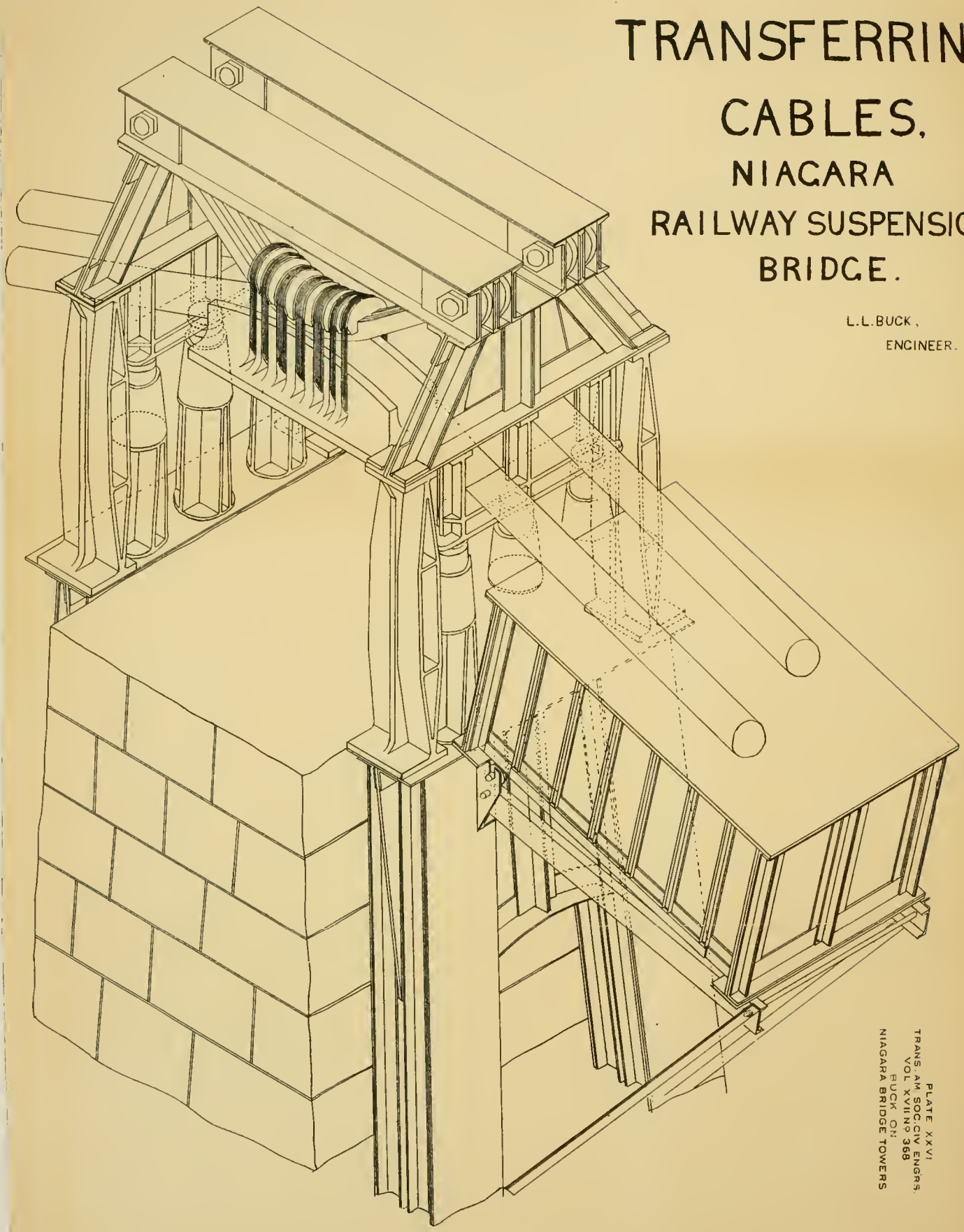
*First.*—The work was done among the columns, rods, struts, clamps and temporary wooden shores.

*Second.*—It was done in freezing winter weather.

*Third.*—Each course was connected with the course below it by several iron dowels.

So far as I could learn no whole stone was found in taking the towers down to the level of the new pedestals, which was as low as they were removed.

In conclusion, I wish to express my satisfaction at the disposition manifested by the gentlemen of the Detroit Bridge and Iron Works to make the workmanship first-class, and also to thank their erecting foreman, Mr. J. H. Stoughtion, for his uniform caution in erecting the iron work and for his valuable assistance while making the transfers.



# TRANSFERRING CABLES, NIAGARA RAILWAY SUSPENSION BRIDGE.

L.L. BUCK,
ENGINEER.

PLATE XXVI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVII No. 368
BUCK ON
NIAGARA BRIDGE TOWERS

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

369.

(Vol. XVII.—November, 1887.)

# REPORT OF PROGRESS OF THE COMMITTEE ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY.

PRESENTED AT THE ANNUAL CONVENTION, JULY 4TH, 1887.*

Your Committee have been able to make some progress during the past six months, but not so much as they had hoped, and beg leave to make a partial report and continue their work, deferring a final report until such time as they feel they have accomplished that for which they were appointed.

Professor Swain, by the aid of two of his students (Messrs. William C. Cushing and Walter F. Thompson), has extended the experiments upon changes of dimension during induration to our American cements. From an extensive memoir, comprising as well their experiments upon a number of other points foreign to our investigation, we condense the following statement. This was partially reported upon at the annual meeting in January, and we omit the description of apparatus which was then given.†

* Committee: Messrs. F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, and A. V. Abbott.

† Proceedings, January, 1887, p. 5.

The cements used were F. O. Norton's Rosendale (marked L); Newark and Rosendale (N); James (E); Hoffman (W); Anchor (T); Standard (V); Taylor's Portland (X); and Keystone Portland (Z).

From each mixture of each cement two 5-inch cubes were made, the mixtures being, 1st, neat cement, 2d, one cement, one sand. Of two of the cements, one cement and three sand were also tried.

The points over which measures were taken by the instrument described in our last report were conical-headed brass tacks driven in after a set of from one hour to one day. It was here that the first difficulty arose, as in a few cases irregularities in measurements were traced to imperfect setting of the tacks.

One specimen of each pair of cubes was left in the air, the other in water.

Both horizontal diameters (the measurements are marked A and B respectively in the table) were measured; the gauge reading taken to the ten-thousandth of an inch (this being the unit in the table of differences appended), and the readings were repeated at each observation from three to eight times.

The intervals of the observations were one day, two days; one, two, four, six, eight and twelve weeks. A careful examination of the corrections, due to unavoidable moderate differences of temperature in the measuring apparatus and specimens at the several observations, showed them to be so small as to cause no change in the result.

An examination of the table warrants the conclusions already reached by other observers, that in general:

*First.*—Cement mortars hardening in air diminish in linear dimensions at least to the end of twelve weeks, and in most cases progressively.

*Second.*—Cement mortars hardening in water increase in like manner but to a less degree.

The experimenters claim that the exceptions to these rules seen in the table are due to inaccurate measurements, the first measurements being in some cases taken too soon, when the tacks were not rigid. Other causes were mentioned which it is not necessary to detail. The changes for the first two weeks are not so regular as for subsequent periods.

*Third.*—The contractions and expansions are greatest in neat cement mortars.

# TABLE No. 1.

Increase or decrease in sides of 5-inch cubes from the first day after mixing up to the age of twelve weeks. The units expressing the differences are each $\frac{1}{10000}$ inch.

+ denotes increase, — denotes decrease.

The first reading in each case is for the neat cement; the second for 1 cement to 1 sand; the third for 1 cement to 3 sand.

| Designation of cement. | Volume cement. | Volume sand. | Per cent. of water. | 1 day in Water. | | 1 day in Air. | | 2 days in Water. | | 2 days in Air. | | 4 days in Water. | | 4 days in Air. | | 1 week in Water. | | 1 week in Air. | | 2 weeks in Water. | | 2 weeks in Air. | | 3 weeks in Water. | | 3 weeks in Air. | | 4 weeks in Water. | | 4 weeks in Air. | | 6 weeks in Water. | | 6 weeks in Air. | | 8 weeks in Water. | | 8 weeks in Air. | | 12 weeks in Water. | | 12 weeks in Air. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B | A | B |
| E | 1 | 0 | 30 | + 4 | −15 | + 7 | +12 | −13 | + 7 | − 2 | − 1 | + 1 | −10 | .... | .... | + 8 | + 5 | − 4 | − 5 | +11 | + 3 | −14 | 0 | +12 | + 7 | −34 | −38 | +26 | +15 | −49 | −50 | +39 | +26 | − 70 | − 65 | +40 | +33 | − 88 | − 86 | + 49 | + 38 | −117 | −119 |
| E | 1 | 1 | 15 | + 0 | + 0 | + 2 | 0 | .... | .... | − 1 | − 4 | + 2 | +15 | .... | .... | +13 | − 6 | + 1 | − 4 | + 6 | −12 | −10 | − 9 | + 5 | + 1 | −13 | −12 | + 6 | − 1 | −21 | −19 | + 9 | − 6 | − 34 | − 32 | −14 | + 4 | − 35 | − 32 | + 12 | − 2 | − 54 | − 48 |
| E* | 1 | 3 | 12 | +32 | + 5 | − 4 | − 9 | −22 | − 2 | − 9 | − 9 | .... | .... | .... | .... | −22 | − 7 | − 9 | − 9 | −10 | − 5 | −13 | −17 | −40 | − 3 | −18 | −19 | −30 | +14 | − 17 | −19 | −40 | +17 | − 16 | − 27 | −47 | +15 | − 17 | − 26 | − 64 | + 11 | − 24 | − 34 |
| T | 1 | 0 | 30 | 0 | − 5 | −19 | − 9 | − 5 | − 7 | −10 | − 1 | .... | .... | .... | .... | − 2 | − 1 | −17 | −15 | + 3 | + 2 | −20 | −15 | .... | .... | .... | .... | +16 | +14 | − 47 | −39 | +34 | +34 | − 71 | − 58 | +39 | +44 | − 83 | − 69 | + 47 | + 47 | −107 | −108 |
| T | 1 | 1 | 15 | + 4 | + 1 | − 4 | + 2 | + 7 | − 2 | .... | .... | .... | .... | .... | .... | +13 | + 6 | + 1 | + 5 | + 9 | + 3 | − 7 | + 1 | .... | .... | .... | .... | +20 | +12 | − 18 | −10 | +26 | +19 | − 27 | − 20 | +30 | +15 | − 32 | − 26 | + 32 | + 15 | − 49 | − 41 |
| T | 1 | 3 | 12 | .... | .... | +23 | +33 | .... | .... | +21 | +41 | .... | .... | .... | .. | .... | | +26 | +39 | .... | .... | +21 | +34 | .... | .... | .... | .... | .... | .... | +16 | +18 | .... | .... | + 16 | + 21 | .... | .... | + 6 | + 12 | .... | .... | + 3 | 0 |
| W | 1 | 0 | 30 | − 6 | 0 | − 8 | + 1 | − 6 | + 5 | − 4 | + 3 | .... | .... | .... | .... | +11 | +15 | + 9 | + 9 | +19 | +23 | + 3 | + 3 | .... | .... | .... | .... | +21 | +25 | − 17 | −22 | +23 | +24 | − 34 | − 33 | +23 | +23 | − 53 | − 59 | + 21 | + 20 | − 84 | − 93 |
| W | 1 | 1 | 15 | ... | .. | 0 | − 5 | + 5 | − 1 | − 1 | 0 | .... | .... | ... | .... | + 2 | + 6 | − 1 | − 1 | +12 | +17 | −10 | − 5 | .... | .... | .... | .... | + 6 | + 9 | − 15 | −10 | +13 | +10 | − 33 | − 38 | + 1 | + 4 | − 41 | − 41 | − 1 | + 7 | − 51 | − 55 |
| V | 1 | 0 | 30 | + 6 | + 6 | + 4 | +12 | +17 | +10 | + 3 | + 7 | .... | .... | .... | .... | +12 | +17 | + 7 | +12 | +12 | +15 | − 8 | − 1 | .... | .... | .... | .... | +26 | +21 | − 44 | −33 | +41 | +34 | − 70 | − 63 | +50 | +44 | − 85 | − 69 | + 57 | + 54 | −116 | −111 |
| V | 1 | 1 | 15 | − 8 | − 2 | + 2 | − 8 | − 4 | − 4 | − 7 | − 5 | .... | .... | .... | .... | 0 | − 2 | − 4 | + 2 | −17 | −14 | −13 | −15 | .... | .... | .... | .... | −12 | − 4 | − 27 | −28 | − 8 | − 2 | − 37 | − 38 | − 4 | + 3 | − 46 | − 45 | + 12 | + 13 | − 50 | − 48 |
| N | 1 | 0 | 27 | + 3 | + 8 | − 9 | + 1 | + 6 | + 2 | −17 | − 9 | .... | .... | .... | .... | +16 | +21 | −22 | −19 | +27 | +25 | −37 | −33 | .... | .... | .... | .... | +48 | +48 | − 79 | −79 | +69 | +69 | −114 | −106 | +88 | +92 | −139 | −118 | +121 | +123 | −145 | −142 |
| N | 1 | 1 | 14 | − 2 | + 3 | − 2 | −13 | + 8 | 0 | − 1 | −12 | .... | .... | .... | .... | −11 | −13 | −14 | −35 | − 3 | − 3 | −20 | −36 | .... | .... | .... | ... | +10 | + 5 | − 36 | −60 | +18 | +18 | − 41 | − 75 | +20 | +20 | − 43 | − 81 | + 37 | + 41 | − 72 | − 85 |
| X | 1 | 0 | 27 to 28 | − 2 | − 4 | + 1 | − 3 | − 1 | − 1 | 0 | + 2 | ... | .... | .... | .... | −10 | − 7 | − 9 | −10 | − 1 | −49 | −12 | −15 | .... | .... | .... | .... | + 5 | + 5 | − 34 | −38 | +16 | +18 | − 48 | − 53 | +16 | +18 | − 54 | − 69 | + 33 | + 39 | − 71 | − 75 |
| X | 1 | 1 | 15 | − 8 | − 9 | − 3 | − 4 | − 4 | − 5 | − 7 | −15 | .... | .... | .... | .... | −18 | − 6 | − 7 | −12 | −10 | − 9 | − 7 | −15 | ... | .... | .... | .... | − 9 | −13 | −16 | −27 | − 9 | − 2 | − 33 | − 42 | − 2 | 0 | − 36 | − 43 | + 4 | + 5 | − 41 | − 41 |
| Z | 1 | 0 | 18 | + 2 | + 2 | − 3 | − 2 | − 5 | +14 | + 5 | + 3 | .... | .... | .... | .... | + 7 | +20 | −14 | −12 | +10 | +17 | −38 | −36 | .... | .... | .... | .... | +23 | +22 | − 71 | −44 | +18 | +26 | − 90 | − 63 | +21 | +31 | −100 | − 71 | + 19 | + 30 | −111 | −103 |
| L | 1 | 0 | 20 | +17 | +19 | − 3 | 0 | +19 | +27 | − 2 | 0 | .... | .... | .... | .... | +38 | +41 | − 3 | 0 | +44 | +47 | −23 | −14 | .... | .... | .... | .... | +47 | +50 | − 61 | −53 | +58 | +59 | − 98 | − 93 | +68 | +65 | −128 | −117 | + 69 | + 70 | −158 | −159 |

* Tack in face A was incorrect.

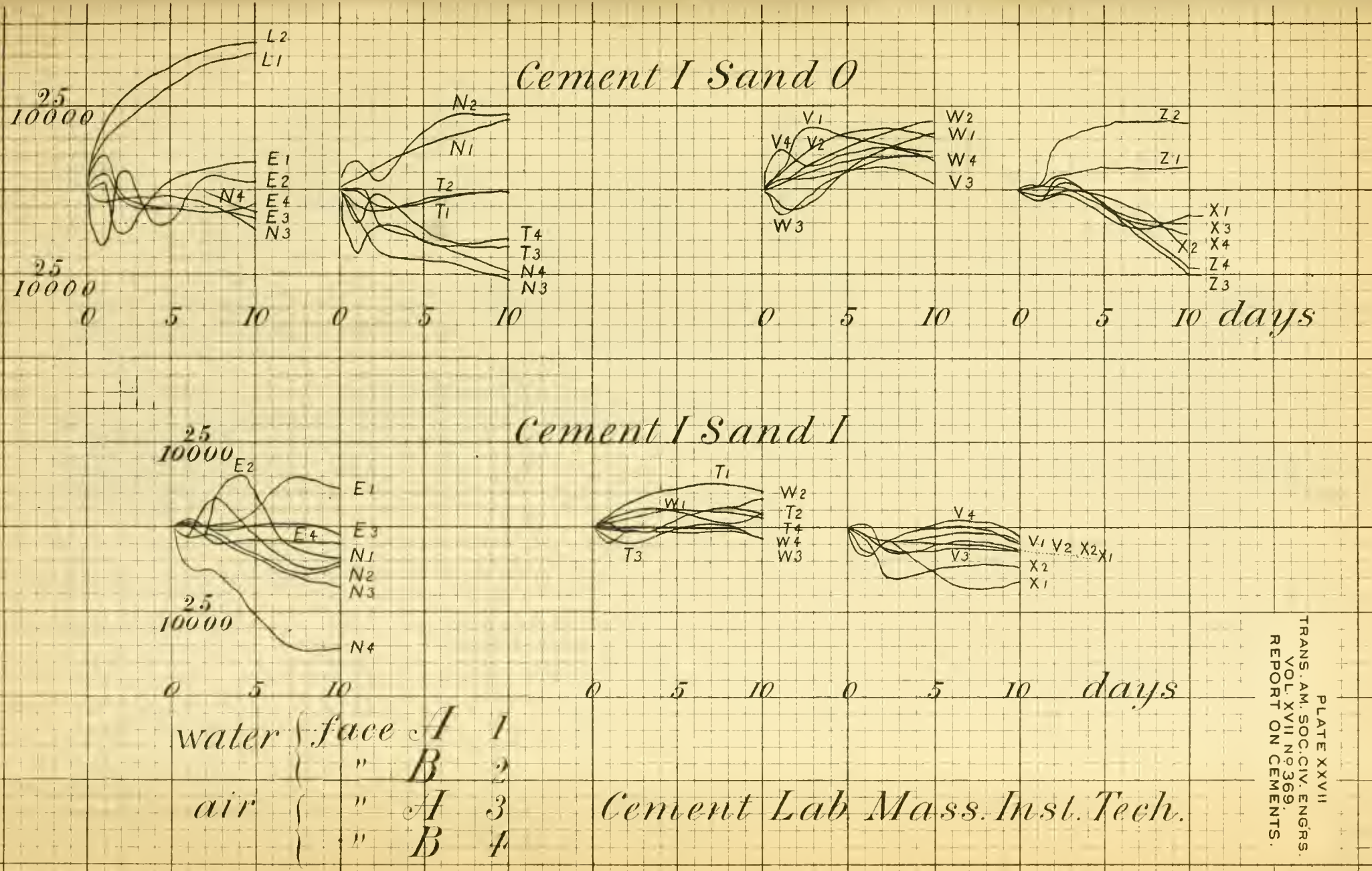



PLATE XXVII
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REPORT ON CEMENTS.

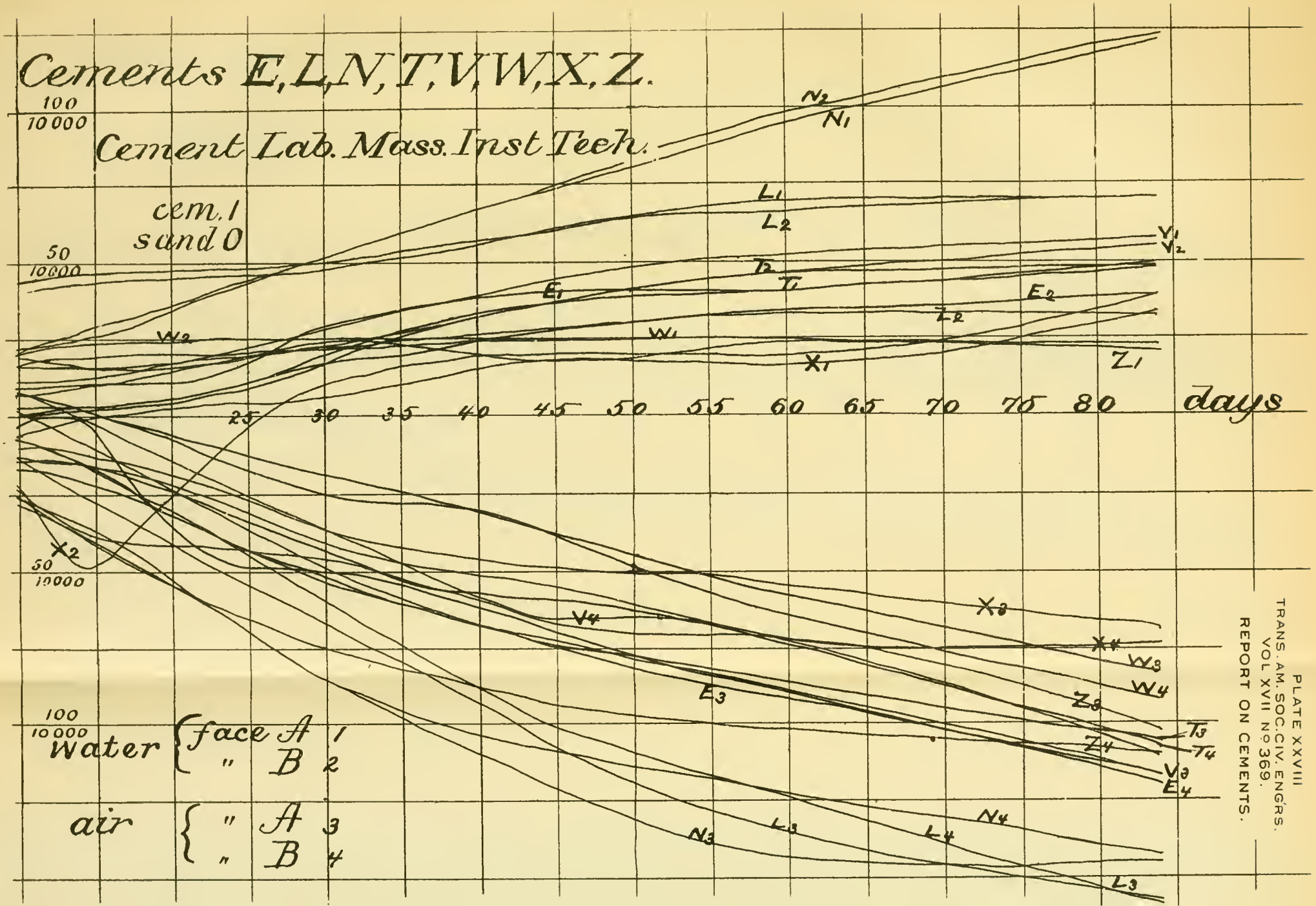


PLATE XXVIII
TRANS. AM. SOC. CIV. ENG'RS.
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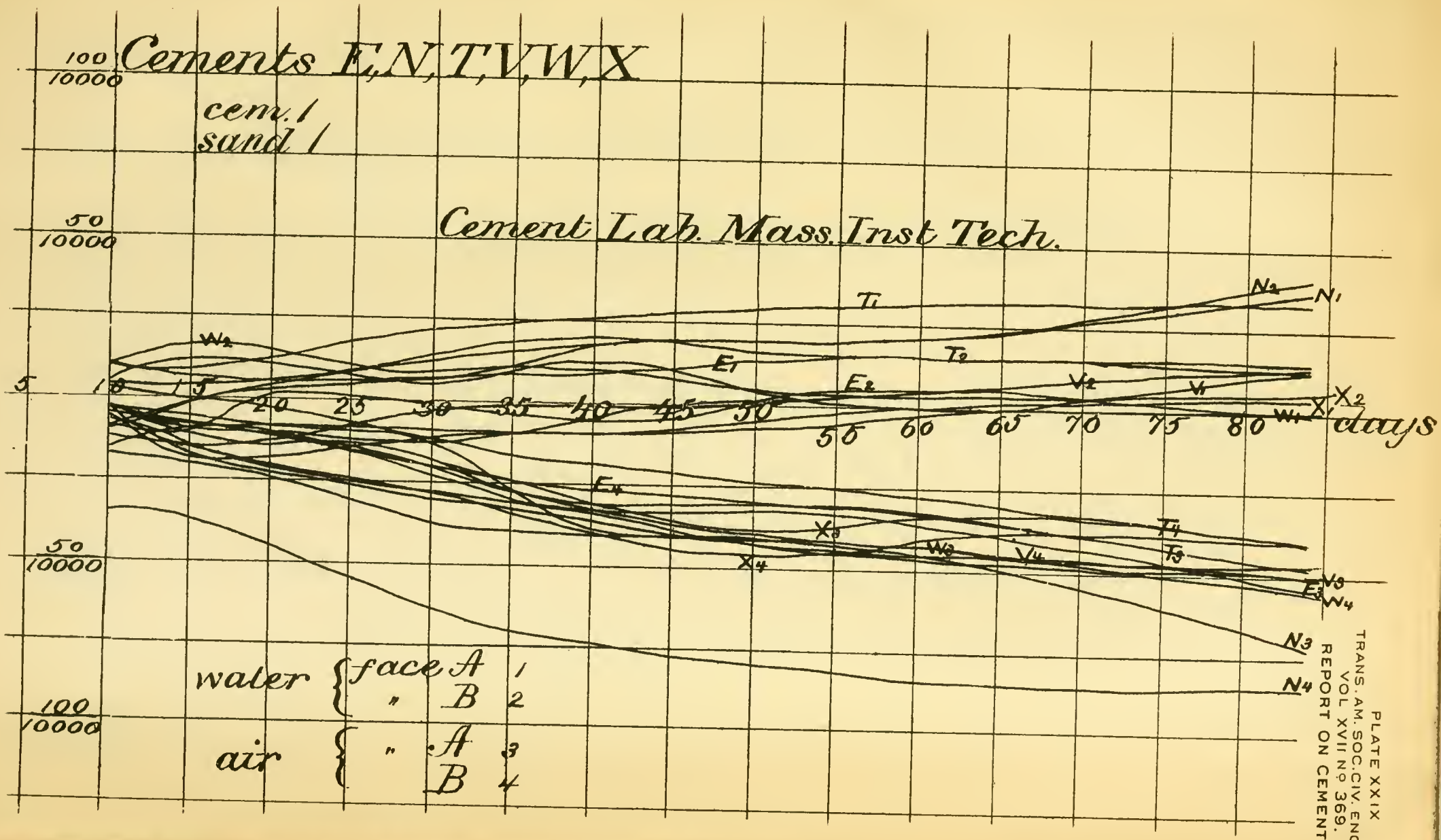


PLATE XXIX
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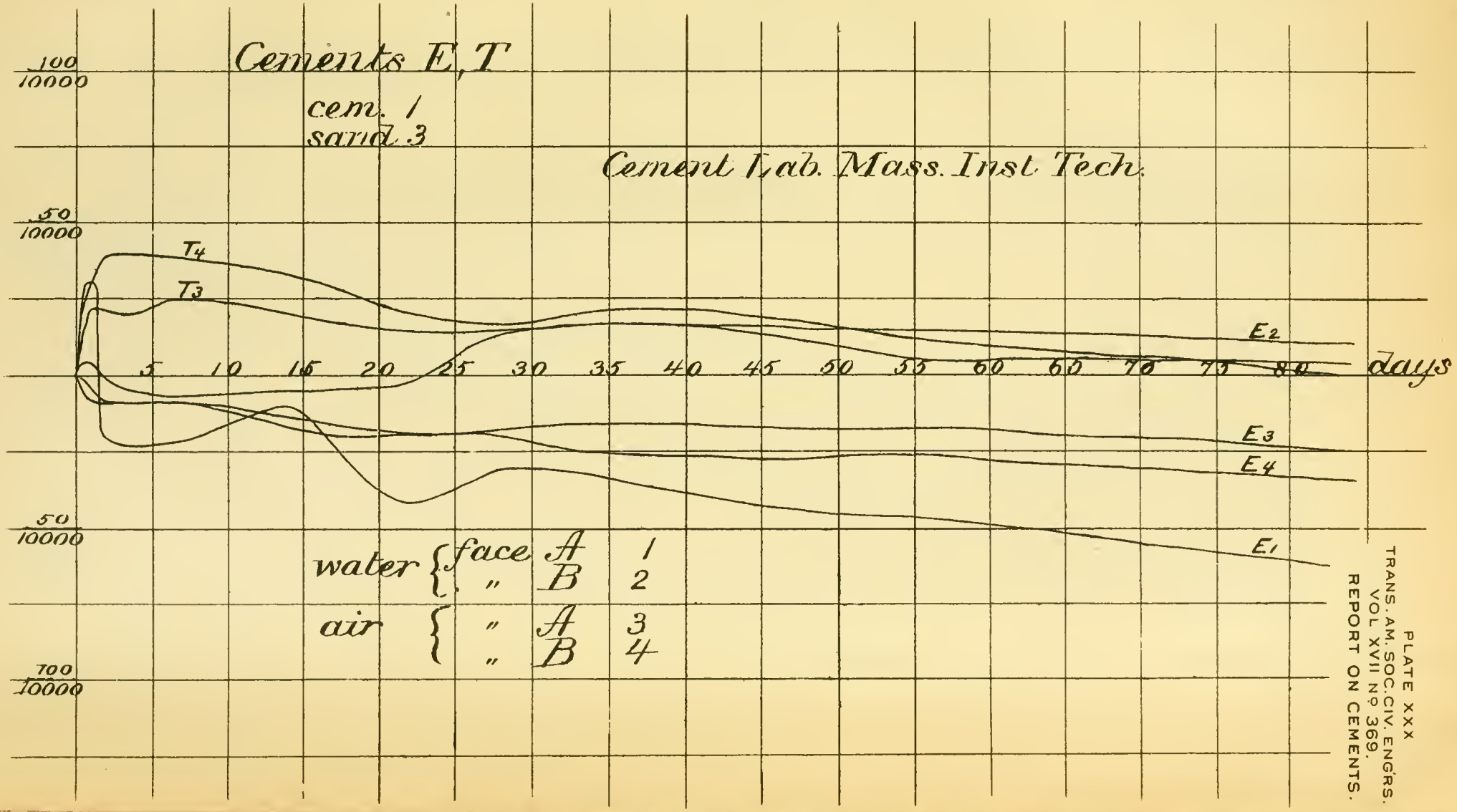


PLATE XXX
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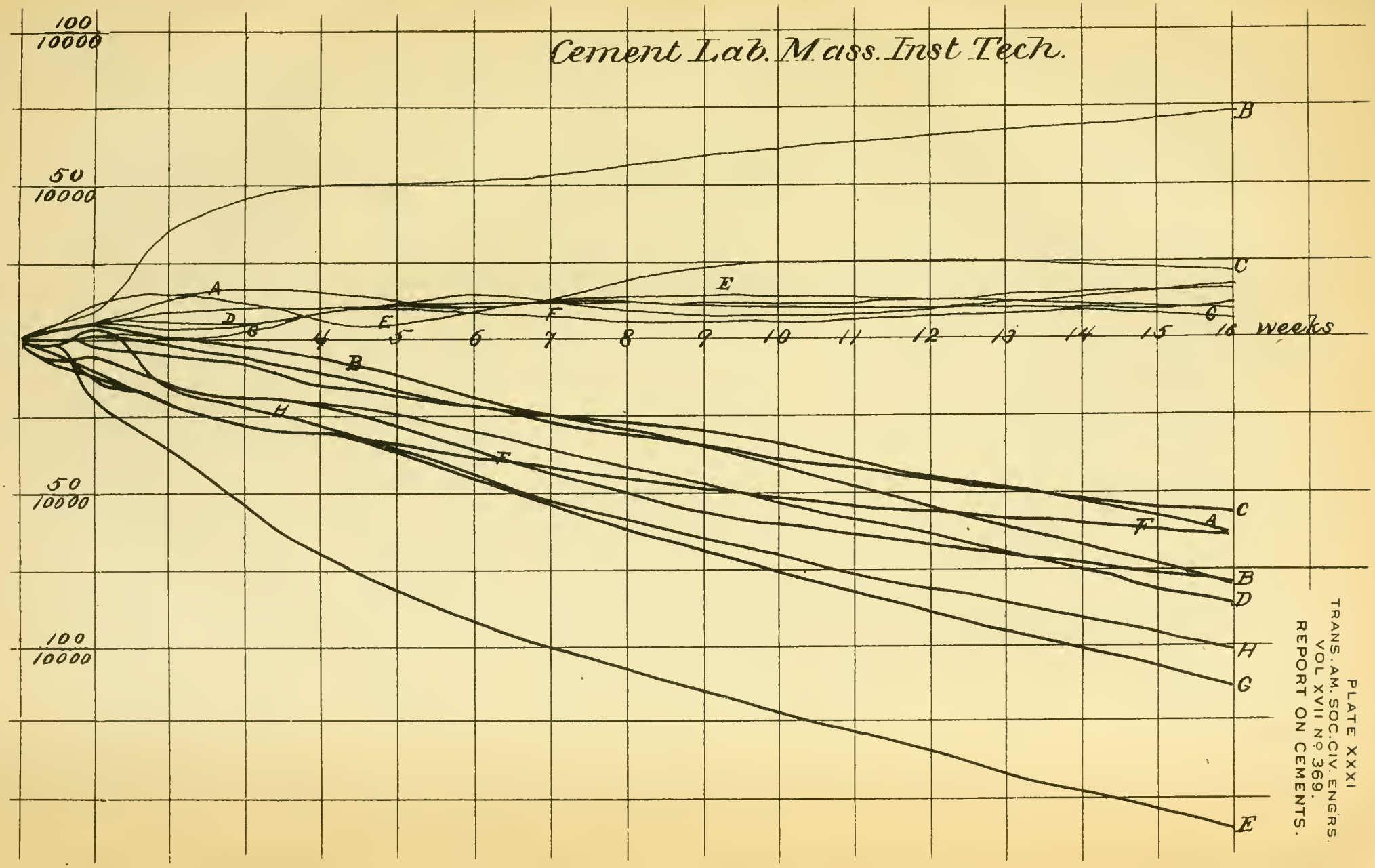


PLATE XXXI
TRANS. AM. SOC. CIV. ENGRS.
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*Fourth.*—The quick setting cements N and L show the greatest contraction when neat. W, which is also quick setting, does not however follow the rule.

The expansion of the quick setting cements is also greater.

*Fifth.*—The changes are less in mortars containing sand.

*Sixth.*—The changes are less in water than in air.

*Seventh.*—The contraction is at the end of twelve weeks:

For neat cement.....0.14 per cent. to 0.32 per cent.
" one cement, one sand ..0.08 " 0.17 "

*Eighth.*—The expansion at the end of twelve weeks is:

For neat cement.....0.04 per cent. to 0.25 per cent.
" one cement, one sand ..0.0 " 0.08 "

*Ninth.*—The contraction or expansion is essentially the same in all directions.

By comparing these figures for American cements with those given by Bauschinger for foreign brands, there is a strking agreement.

He gives (see our previous reports) for neat cement, contractions in air in sixteen weeks of 12 to 34 per cent., and for one cement to three sand, 8 to 15 per cent. For expansion of neat cement in water in sixteen weeks 1 to 15 per cent., and for one cement to three sand, 0 to 2 per cent. The expansions being less in his experiments. The experimenters on our American cements however, consider the 25 per cent. expansion (Specimen N in the table) as probably erroneous, for reason given by them.

Comparing the average co-efficients of expansion, we have:

For nine cements, neat, by Prof. Bauschinger.. 0.00048 in 16 weeks.
" English Portland, neat, by Mr. Grant..... 0.00084 " 3 months.
" American cements, neat, by present table... 0.00105 " 12 weeks.
" " " omitting the one claimed in error..... 0.00085 " 12 "

agreeing closely with Mr. Grant's.

Plates XXVII to XXX have been prepared from the table to illustrate the subject graphically, and the Plate XXXI shows the same for Professor Bauschinger's table.

Mr. Maclay, of the committee, has sent a few results of experiments on absorption which are of interest. From accurate weighings he gives the following table of gains in weight in perfectly dry mortars after immersion in fresh water for the times stated.

| Cement. | Sand. | 12 hours' immersion. | 36 hours' immersion. | 7 days' immersion. | 79 days' immersion. |
|---|---|---|---|---|---|
| Rosendale.....1 | 2 parts. | 11 per cent. | 11 per cent. | 11 per cent. | 15.40 per cent. |
| " .....1 | 1 part. | 11 " | 11 " | 11 " | 15.06 " |
| Portland .....1 | 3 parts. | 9.2 " | 10.9 " | 10.9 " | 15.08 " |
| " neat.....1 | 0 part. | 11.2 " | 11.2 " | 12.06 " | 13 08 " |

The mortars, it is seen, gained less than the neat Portland up to seven days, after which the neat Portland absorbs least.

Mr. Maclay also sends some results of compression tests on cylinders 24½ inches long and 4⅛ inches diameter.

TABLE No. 2.

Compression tests of three cylinders 24½ inches long and 4⅛ inches in diameter (area 13 $\frac{36}{100}$ square inches) made from (Hammil & Gillispie's) Portland cement.

| Neat Cement, 104 days old. | | | 1 Cement, 2 Sand, 3 mos. old. | | 1 Cement, 3 Sand, 2 mos. old. | |
|---|---|---|---|---|---|---|
| Load lbs. sq. in. | Comp. inches. | Differences. | Compression inches. | Differences. | Compression inches. | Differences. |
| 100 | 0.0237 | | | | 0.0062 | |
| 150 | .0250 | 13 | 0.0010 | | .0095 | 33 |
| 200 | .0267 | 17 | .0023 | 13 | .0135 | 40 |
| 250 | .0283 | 16 | .0035 | 12 | .0175 | 40 |
| 300 | .0300 | 17 | .0050 | 15 | .0220 | 45 |
| 350 | .0323 | 23 | .0073 | 23 | .0253 | 33 |
| 400 | .0352 | 29 | .0097 | 24 | .0290 | 37 |
| 450 | .0375 | 23 | .0115 | 18 | .0330 | 40 |
| 500 | .0402 | 27 | .0142 | 27 | .0383 | 53 |
| 550 | .0427 | 25 | .0168 | 26 | .0433 | 50 |
| 600 | .0450 | 23 | ·0190 | 22 | .0480 | 47 |
| 650 | .0473 | 23 | .0210 | 20 | .0567 | 87 |
| 700 | .0495 | 22 | .0230 | 20 | .0653 | 86 |
| 750 | .0515 | 20 | .0270 | 40 | 750 pounds crushed. | |
| 800 | .0535 | 20 | .0290 | 20 | | |
| 850 | .0553 | 18 | .0310 | 20 | | |
| 900 | .0570 | 17 | .0330 | 20 | | |
| 950 | .0587 | 17 | .0367 | 37 | | |
| 1 000 | .0600 | 13 | .0403 | 36 | | |
| 1 050 | .0610 | 10 | .0450 | 47 | | |
| 1 100 | .0630 | 20 | 1 125 pounds crushed. | | | |
| 1 150 | .0643 | 13 | | | | |
| 1 200 | .0650 | 7 | | | | |
| 1 250 | .0677 | 27 | | | | |
| 1 300 | .0697 | 20 | | | | |
| 1 350 | .0710 | 13 | | | | |
| 1 400 | .0737 | 27 | | | | |
| 1 450 | .0753 | 16 | | | | |
| 1 500 | .0760 | 7 | | | | |
| 1 550 | .0770 | 10 | | | | |
| 1 600 | .0777 | 7 | | | | |
| 1 650 | .0780 | 3 | | | | |
| 1 700 | .0793 | 13 | | | | |
| 1 750 | .0813 | 20 | | | | |
| 1 800 | .0840 | 27 | | | | |
| 1 850 | .0847 | 7 | | | | |
| 4 358 | Crushed. | | | | | |

The first test in the table is of neat cement at a little over three months old. The results, it will be seen, are somewhat irregular, and no one point can be assigned as a limit of elasticity.

On the contrary, the compressions for equal increment of strain grow gradually less.

At 200 pounds per square inch the co-efficient of compression per 100 pounds of load is 0.00054; at 500 pounds it is 0.00033; at 700 pounds it is 0.00029; at 1 000 pounds, 0.00024; at 1 500 pounds, 0.00021; and at 1 850 pounds, or just before crushing, it is 0.00019.

Examining the similar prism mixed of one cement and two sand, we find co-efficient at 200 pounds is 0.000047; at 500 pounds, 0.000116; at 700 pounds, 0.000138; at 1 000 pounds, 0.000164; the specimen crushing at 1 125 pounds.

Here we see the change is in the opposite direction, gradually increasing; beginning with less than one-tenth of that of neat cement, and at 1 000 pounds being two-thirds of that of neat cement.

Examining the prism mixed three to one, and two months old, we get at 200 pounds, 0.000276; at 500 pounds, .000313; at 700 pounds, 0.000381; being one-half that of the neat cement at 200 pounds, and one-third greater at 700 pounds.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVII.—November, 1887.)

# PORTLAND CEMENT TESTING.

BY HENRY FAIJA, M. Inst. C. E.

READ DECEMBER 1ST, 1886.

The object of testing cement is to determine its constructive value, while the necessities of a test are that it should be expeditious, complete, and reliable. That all tests should be carried out in a similar manner is desirable, so that the comparative value of different cements may be determined. Most large users of cement have each a system peculiar to themselves for carrying out a cement test, so that the results they obtain are of value to themselves as comparative tests. Each of these therefore acknowledges indirectly the value of a uniform system of

These remarks on Cement Testing have been suggested by the Report of the Committee on a Uniform System for Tests of Cement, appointed by the American Society of Civil Engineers, (Trans., Vol. XIV.) They can scarcely be considered a criticism, nor can they be taken as forming any part of a discussion on any of the points mentioned in the report; for, taken as a whole, the author agrees with the suggestions and advice given therein. They are really only a *résumé* of the author's system of carrying out a cement test, a system which he has adopted and developed, after many years' experience in testing cement, as being simple, expeditious, and enabling a just estimation being arrived at of the value of the sample under consideration for constructional purposes, and as such they may throw a little additional light on a subject which the committee have already so fully and ably dealt with.

carrying out a test. Now, clearly, that which is good for and of use to the individual, must be of equal use to the multitude, *i. e.*, the whole of the professions and trades that are connected with the cement industry, whether as buyers, sellers, makers, or users. In fact it is generally acknowledged that a uniform system for carrying out a cement test is a desirable object to attain. But there seem to be so many prejudices to be overcome, both among users and manufacturers, for the systems which they have each individually adopted, that at all events in England it does not seem at all likely that any uniform system will ever be adopted, though in course of years the *rationale* of a cement test may be more generally understood and appreciated, in which case it may be expected that tests will be carried out on somewhat approximate lines.

In most countries on the Continent of Europe a uniform system has been devised and put in force by the respective governments, and the report of the committee appointed by the American Society of Civil Engineers to investigate the matter is a paper which deserves the most serious and careful consideration of all persons interested in the subject. Without giving a specification, it supplies all the information necessary to enable each user to draft such a specification as will suit his special requirements, and gives most carefully considered and valuable instructions for the guidance of those who have to carry out the test. If anything in the report is to be regretted, it is the modesty with which the committee put forward, as recommendations only, the result of their labors.

There are one or two points in the report which invite discussion, notably where it is suggested that the quality of a cement may, to a certain extent, be determined by the brand on the outside of the barrels. A maker may have a great reputation, and may, as a rule, produce good cement, but it is out of all reason to suppose that the quality of his manufacture does not occasionally vary. In the author's opinion the value of any lot of cement should be determined by properly taken samples, fairly representing the delivery, and that its value should be determined by the absolute results obtained in testing, independently of brand or other knowledge respecting the manufacture.

The value of a cement may be determined by its fineness, and its tensile strength at two dates. The greater or lesser increase in strength between the two dates defines the growth, and determines approximately

the ultimate strength which may be expected from it. In practice, a test at three days from gauging, and another at seven days, enables a sufficiently accurate determination to be arrived at. The greater the increase in strength shown between those two dates, the longer it may be expected that the cement will continue to increase in strength, and the greater the ultimate strength obtained. To make this clearer, the results of tests of twelve different cements are given in Table No. 1 in the Appendix, whereby it will be seen that the increase between the three and seven days in all cases fairly indicates the strength which will be developed at the longer dates.

The actual strength demanded of a sample, and whether it shall be quick or slow setting cement, must of course be determined by the special requirements of the work. But clearly a quick setting cement, which shows but a small increase between its strength at three and seven days, and which will probably acquire its greatest strength in a short time, should develop a greater strength at the seven days than a slower setting one, which may continue to increase in strength for a considerably longer period.

It may be here added that quick setting cements as a rule attain their greatest strength in about six months; after then there seems to be a slight falling off in strength, but to what cause this is due has yet to be determined, as there seems no reason, theoretical or practical, why a strength once attained should be partially lost.

The adhesive test is unquestionably an uncertain and untrustworthy one. The adhesion of a cement to any material depends as much on the nature of the other material as on the cement, and even where seemingly similar surfaces are secured, it is impossible to be sure that there may not exist slight differences which will affect the result obtained.

The same argument, in the author's opinion, applies to what is known here as the sand test, with the further additional disadvantage that a considerably greater amount of skill is required to gauge a sand briquette than to gauge one of neat cement. The ordinary practice for carrying out a sand test here is by gauging one part of cement with three parts of standard sand by weight, using as little water as practicable; and the usual specification is that briquettes gauged in this manner shall be placed in water in twenty-four hours, where they shall remain until due for testing, and that they shall be tested at twenty-eight days from gauging, and carry a tensile strain of two hundred pounds.

The standard sand used here is a clean white sand which is found at Leighton Buzzard. It is washed and sifted, and only that which is retained between a 20 and 30-mesh sieve is used for testing. It is perhaps the best sand which can be readily obtained for the purpose, but that it is absolutely constant and uniform of grain and nature is not to be expected. Certainly a sand test made with other sand than this, though sifted to the required degree of fineness, would not give similar results.

The principal objection to the sand test however, is the difficulty of manipulation. It is hardly skill and knowledge of cement which enables a manipulator to obtain a good result from a sand briquette; it seems to be a peculiar knack, impossible to teach, which some possess, and others can by no amount of practice acquire. It is further questionable if the sand test gives any better guide as to the value of the cement for practical purposes than the three and seven days' neat test, for it will readily be admitted that a standard sand is an impossible sand to use in practical work.

The fineness suggested in the report is the same that has been adopted by the author, viz., ten per cent. on a sieve having 2 500 holes to the square inch, it being as much as can be expected of the manufacturer, unless an additional price is paid for extra grinding, as modern machinery will not allow of a good hard clinker being economically ground to a greater fineness than this. And it seems to be rather a question for the user, should he require a greater degree of fineness, whether it is more economical to pay the manufacturer an extra price for finer grinding, or use more cement in proportion to his aggregate.

The most important point however, is in the first instance to determine the soundness of a cement, *i. e.*, absence of "checking or cracking," or what is called in this country "blowing." Given that a cement shows that it will at some future day blow, it is evidently of no value, no matter to what degree of fineness it may be ground, or what tensile strength it may develop within the limits of time of a test. Many cements will give good results at the three and seven days, and even sometimes at the twenty-eight days, but will eventually "blow," and the ordinary means of determining the soundness of a cement are, in the author's opinion, insufficient. For this purpose the author devised an apparatus which he calls a steamer, and which, through the kindness of Mr. T. C. McCollom, M. Am. Soc. C. E., has been presented to the society. The apparatus is strictly accurate in theory. The causes of

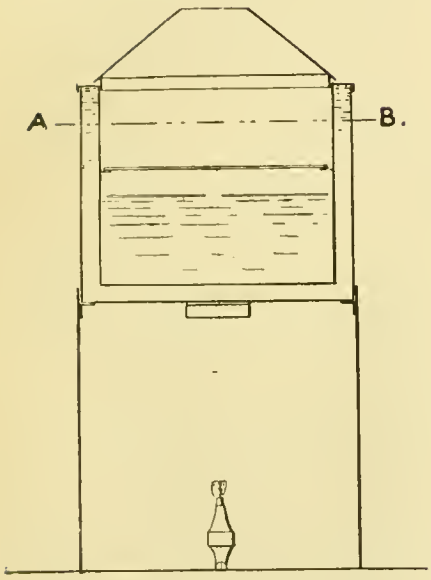
"blowing" in the cement are that it may have an actual excess of lime in its composition; or it may be that the manufacture in one or other of its stages has not been properly carried out. But in all cases the blowing is due to a portion of the lime being uncombined with the silica and alumina, and the action which takes place in the apparatus is either the rapid development of the setting properties, if the cement be sound; or, in the case of the cement being unsound, the rapid development of those characteristics which render it so.

The apparatus has been in constant use in the author's laboratory for the last four years, and he is pleased to be able to say that once or twice when he has ignored the warning given by the apparatus he has found himself in error.

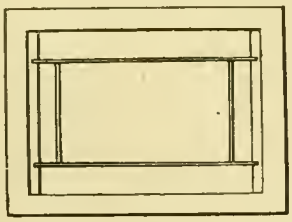
The author generally makes three pats on pieces of glass or other non-porous substance. Pat No. 1 may be left in the air for the determination of color, if such is of any value. Pat No. 2 is put in water as soon as it is considered set; this is more to determine the time of set than for any other purpose. Pat No. 3 is treated in the apparatus, a sketch of which is given in Fig. 1, Plate XXXII. It consists of two vessels, one within the other, a water space being thus maintained between them, which assists in equalizing the temperature of the inner or working vessel, which vessel contains water up to the level shown, and above the water are racks on which the pats are first placed. By means of a gas jet the water in the inner vessel is maintained at an even temperature of 110 degrees Fahr., the space above the water being therefore filled with vapor rising therefrom, which is at a temperature of about 100 degrees Fahr. The apparatus is covered, and through the cover a thermometer is inserted, so that any rise or fall in temperature may be immediately observed and corrected by raising or lowering the gas jet.

Directly the pat is gauged it is placed on the racks in the upper part of the vessel. It will be found that even very slow setting cements will, in this moist heat, be perfectly set in a few hours; it is however usual to leave the pats on the racks for six or seven hours, when they are removed from the racks and placed in the warm water. This time however is only a matter of convenience, as by this arrangement pats ganged in the morning are left on the racks all day, and are placed in the water before leaving in the evening. They are left in the water (the temperature being maintained throughout the night), and are examined the next morning. If the pats are then perfectly sound, the author has no hesitation in saying that the cement is a perfectly sound and safe one to use, and the briquettes are then made to ascertain whether the sample complies with the other conditions of the specification. Should any indications of blowing appear in the pat, the cement is laid out in a thin layer to cool, as it is possible that the cement may have blown through

Fig 1.

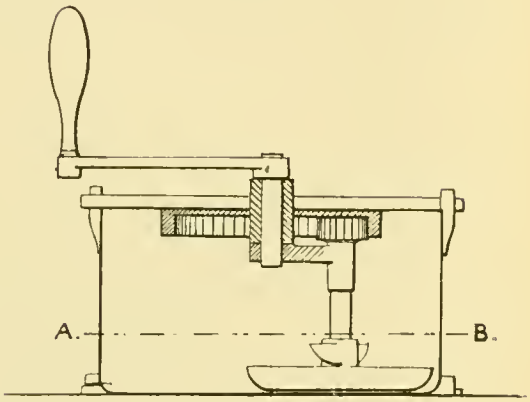



Sectional Elevation.

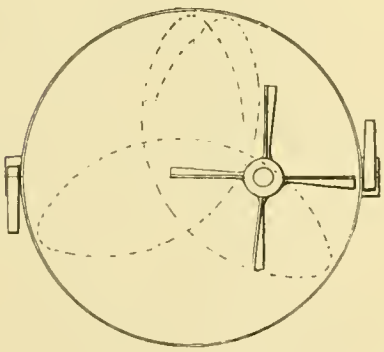


Plan at A.B.

Fig 2.




Sectional Elevation.



Plan at A.B.

PLATE XXXII.

being over fresh, and that a few days' cooling would render it sound. If, however, pats made from cement which has been cooled for three or four days still blow, it is absolutely certain that the cement is an unsound one, and will at some future day blow and destroy the work in which it has been used. The great advantage of this test is that it enables the soundness of a cement being determined with certainty in a few hours.

In manipulating cement for testing, the author finds mechanical means infinitely superior and more constant in results than hand-gauging. The apparatus which he uses is one which he patented some years ago. It simply consists of stirrers revolving round their own axis in one direction, and round the pan in the reverse. Fig. 2, Plate XXXII, shows generally the appearance of the machine. The author always gauges five briquettes at a time, as by that means a better average result is obtained than when briquettes are gauged separately. For this purpose he has the five moulds fixed together on a single bed plate. The five moulds require thirty ounces of cement to fill them. The stirrers are removed from the gauger and the cement put in. The stirrers, which are fixed to the top framing of the machine, are then replaced, and the required quantity of water added; the handle of the machine is turned for a minute or so until the cement gives indications of balling. The stirrers are then again removed, the pan taken off the bench, and the cement knocked up with a spatula to one side of it, and turned out on to the gauging slate. It is then knocked up with a trowel into a convenient form, and the five briquette moulds filled and lightly rammed in the usual way to remove air bubbles. Sand briquettes are gauged in an exactly similar manner, beyond that the sand and cement are mixed dry in the machine before the water is added.

A matter which is of great importance in obtaining the best results in testing, is that the minimum of water should be used in gauging. The actual amount of water of crystallization found in a perfectly hard and set cement is only about ten per cent. It is therefore evident that all water used more than this is in excess, but it would be impossible to manipulate with such a small quantity, added to which different cements require different quantities of water to enable them to be properly gauged. This the author always determines by experimental pats made previously to gauging the briquettes for testing.

It is with a very great deal of satisfaction that the author finds that your committee recommend in their report that the rate of speed at which the strain should be put on to a briquette when being tested is 400 pounds per minute, it being the same which he has adopted for some time, and which he recommended in a paper presented to the Institution of Civil Engineers here three years ago.* The only reason he had in fixing this speed was that it was a convenient one; but it is

* See Proceedings Institution of Civil Engineers, Vol. LXXV, page 225 and following.

most important that a standard rate of speed should in all cases be adopted. In the paper above referred to, he gave the results of several hundred experiments he had made in applying the strain at different rates of speed. The different speeds at which he worked were:

| | | |
|---|---|---|
| 100 pounds in 1 second. | 100 pounds in 30 seconds. | |
| 100 " 15 seconds. | 100 " 60 " | |
| 100 pounds in 120 seconds. | | |

and the results obtained are shown in Table No. 2 in Appendix.

Taking the slowest speed as zero, it will be seen that an increase of nearly twenty-five per cent in strength is obtained by adopting the quickest.

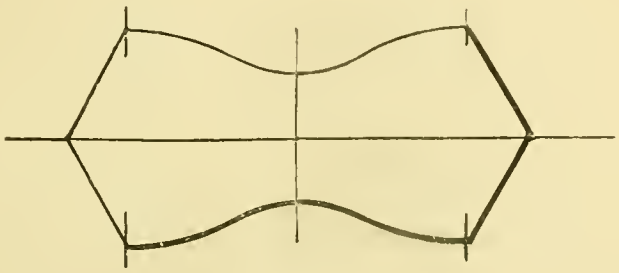


FIG. 3.

Fig. 3 gives the form of briquette adopted by the author, which seems similar in form to that recommended by your committee. So far as testing machines are concerned, there is nothing much to say. They are simply weighing machines, and that one which is the most convenient to use, the least liable to error, the easiest to check, and the most economical in cost, is the one which is probably the best to adopt. The one which the author has devised and patented complies with most of these conditions, and as he has had the honor of the acceptance of one by your society, it can be seen by any of its members at the society house.

The specification which may be deduced from the foregoing remarks is as follows:

1. *Fineness.*—To be such that the cement will all pass through a sieve having 625 holes ($25^2$) to the square inch, and leave only ten per cent. residue when sifted through a sieve having 2 500 holes ($50^2$) to the square inch.

2. *Expansion or Contraction.*—That a pat made and submitted to moist heat and warm water, at the temperatures and in the apparatus hereinbefore described, shall show no signs of blowing in twenty-four hours.

3. *Tensile Strength.*—Briquettes which have been gauged, treated and tested in the prescribed manner, to carry an average tensile strain, without fracture, of at least 176 pounds at the expiration of three days from

gauging; and those tested at the expiration of seven days from gauging to show an increase of at least fifty per cent. over the strength of those at three days, but to carry a minimum of 350 pounds per square inch.

It will be found that this specification satisfies most requirements of a slow setting cement. If a quick setting cement is wanted, a greater tensile strength (say 400 pounds) might be asked for at the seven days, but the increase between the three and seven-day tests should be reduced to say twenty or twenty-five per cent., but would naturally vary in accordance with the actual quickness of setting developed by the cement.

It may be thought that the tensile strengths above specified are low, but to ask for a greater strength would be asking for that which, except under special conditions, could not be always safely attained. Very high tensile strengths at early dates generally indicate a cement verging on an unsound one. It must also be remembered that a manufacturer will always try to be well on the safe side with the tensile strength, for he not only has to comply with the specification, but he has to run the risk of his cement being gauged and tested by inexperienced persons, who would probably be more likely to condemn the delivery than others of greater experience. In specifying therefore 350 pounds, the user may be pretty sure that he will obtain a cement carrying nearer 400 pounds, which, for a slow setting cement, is certainly ample to obtain at seven days; while with the quick setting cements instead of 400 pounds, probably 450 or 500 pounds would be the tensile strength developed.

In conclusion, the author cannot express too strongly his opinion of the desirability and importance that a universal and standard system of testing cement should be adopted, in order that results obtained in testing may be of value and for comparison. More especially does this apply between two countries like England and the United States, where a large quantity of English manufactured cement is used. A test is made here and certain results obtained. If a different procedure is then adopted on its arrival in the United States, different results are obtained, and unsatisfactory arguments follow. And there is no doubt that the thanks of all makers and users of cement in the two countries are due to the committee appointed by your society for the very serious and careful attention which they have given to the subject, and the lucid way in which their report is published, and it is only his full conviction of the importance of this subject which has emboldened the author to address your society on the subject.

# APPENDIX.

TABLE No. 1.

| No. | 3 days. | 7 days. | 28 days. |
|---|---|---|---|
| | Pounds. | Pounds. | Pounds. |
| 1 | 360 | 406 | 428 |
| 2 | 340 | 388 | 403 |
| 3 | 400 | 448 | 452 |
| 4 | 342 | 350 | 395 |
| 5 | 480 | 551 | 580 |
| 6 | 590 | 658 | 666 |
| 7 | 203 | 381 | 504 |
| 8 | 231 | 420 | 548 |
| 9 | 305 | 515 | 676 |
| 10 | 196 | 387 | 595 |
| 11 | 255 | 518 | 692 |
| 12 | 202 | 458 | 585 |
| 13 | 67 | 312 | 604 |
| 14 | 82 | 380 | 540 |

Nos. 13 and 14 are both cements that were not set in twenty-four hours, and though they can scarcely be considered fair cements to take as examples of a principle, they undoubtedly lend considerable weight to the argument. The results obtained are very peculiar, more especially in the case of No. 13, where the seven days' test is five times greater than the three days, and the twenty-eight days twice the seven days.

TABLE No. 2.

| | | | | | |
|---|---|---|---|---|---|
| 100 | pounds in | 120 | seconds | .....400 | pounds. |
| 100 | " | 60 | " | .....415 | " |
| 100 | " | 30 | " | .....430 | " |
| 100 | " | 15 | " | .....450 | " |
| 100 | " | 1 | " | .....493 | " |

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

371.

(Vol. XVII.—November, 1888.)

# THE VENTURI WATER METER: AN INSTRUMENT MAKING USE OF A NEW METHOD OF GAUGING WATER; APPLICABLE TO THE CASES OF VERY LARGE TUBES, AND OF A SMALL VALUE ONLY, OF THE LIQUID TO BE GAUGED.

By CLEMENS HERSCHEL, M. Am. Soc. C. E.

READ DECEMBER 21ST, 1887.

"Induction; analogy; assumptions founded on facts and unceasingly rectified by additional observations: a genial form of tact, inborn, but strengthening itself by making numerous comparisons between its indications and the results of experiment; such are the principal means of arriving at the truth."—Laplace, *Des divers moyens d'approcher de la certitude.*

Some additional instrument or method for gauging water has long been desired by hydraulic engineers. In the case of water flowing through pipes, as in city water-works, it is extremely difficult or impracticable to meter the water, as soon as diameters approaching one foot, or quantities approaching one million gallons daily, are reached. In some such cases the stream of water has been split up into many smaller ones, each of which was then furnished with a meter, and the tail water of these meters reunited—a method and apparatus so cumbersome and costly as to be rarely applicable. Taking the case, on the

other hand, of a far less valuable commodity, viz., of water under little or no pressure, about to be or after it has been used for power, the practical difficulties of gauging again become very great. Ordinary meters are out of the question, owing to the small value of the article per cubic foot, and to the proportionately great cost, per cubic foot of water metered, of applying a mechanical meter. Still, an approach to metering the water is found, when the turbine or other water motor is converted, by previous tests or experiments, into a form of water meter; and this method is doing excellent and extensive service for many water-power companies. It has its encumbrances, as, that the wheel must be sent to a "testing flume" before it is set, unless it can be tested after setting, at a cost which in most cases is prohibitory. There is also some water used, as a rule, in manufacturing establishments, which does not pass through the wheel. And in mills that use cheap wheels, and change their wheels frequently, it causes considerable work and expense to maintain the water wheel system of measurements.

The weir is seldom permanently applicable to such cases, on account of the difficulty and expense of setting and maintaining it, and because it diminishes the head acting on the wheel. If put up-stream from the wheel, it must take the form of a movable weir, sliding up and down in grooves. The weir edge is then raised or lowered until the quantity passing the weir is just equal to the draft of the wheel; a delicate operation, and applicable only in the case of experiments, where the engineer has full control of the wheel as well as of the rest of the apparatus, and as a last resort. Possibly this apparatus has a future yet before it for restricting the use of water in those cases where some slight sacrifice of head may be made, and where an overdraft on the part of any water-taker may be allowed to bring about its own punishment, in the shape of a material lowering of the water-level in the penstock, but without causing other damage. When not needed for purposes of restriction or of metering, it could be lowered out of the way sufficiently to cause scarcely any obstruction to the draft of water; or if at other times so desired, it could be raised sufficiently to act as a head-gate. Ice in the winter time would make the sliding of the weir portion in its grooves somewhat troublesome, but that could be overcome as it is overcome in the case of similar appliances. Perhaps the greatest objection to the apparatus in actual practice is, that it acts so harshly against the consumer, when moral suasion, with constraining forces held

in reserve, would be more advisable. As limiting the draft of water, it is better, than for purposes of metering. Like many another device, it has its advantages and its disadvantages.

If the weir be put in its usual position, down-stream from the wheel, the ordinary construction of tail-races, which are generally designed to meet the wants of musk-rats rather than those of hydraulic engineers, becomes a serious or insuperable obstacle to the use of this apparatus. In some few cases there is such an abundance of fall that a permanent weir can be set in the tail-race, but the general rule permits of weirs only during experiments made on wheels after they are set, and to a portion only of these distinct cases.

To establish with accuracy an economical method for gauging large quantities of flowing water, Mr. James B. Francis, Past President Am. Soc. C. E., some thirty years ago made experiments with floating tubes, in rectangular flumes, as recorded in his "Lowell Hydraulic Experiments," third edition, 1871. And his method has since then done good service in many instances. It also has, however, its limitations; requiring a straight, open, rectangular flume, of considerable length, in which to operate.

Another method is the one which uses the current-meter, an example of whose best work is described in "Boston Water-works, Additional Supply," by A. Fteley, M. Am. Soc. C. E. Its limitations consist in delicacy of the instrument, which renders it nigh hopeless to expect good results, or even to save the instrument whole, when it is put into the hands of the average engineer's assistant; and in the fact that instrument makers have never yet been brought to the point of selling current-meters rated and ready for use; while, on the other hand, for each one who wishes to use the current-meter, to build and maintain a rating apparatus becomes impracticably onerous and expensive.

Still another method lies in the application of the Pitot tube, and of its modified forms, one of which makes use of it simultaneously at many points distributed over the cross section of a closed or other conduit. And there are other instruments described in hand-books and in current literature on the subject.

It has long seemed to the writer that an application to metering water of the principle involved in the Bourdon anemometer, an instrument which has been used to measure the velocity of currents of air in mines, in France, would yield valuable results, and the present paper is in-

tended to record the experiments made and the results found with two sizes of water meter of that description. Bourdon's anemometer is founded upon the property of a Venturi tube to exercise a sucking action through holes bored into its narrowest section. Then by measuring the intensity of this aspiration by means of any form of vacuum gauge, and establishing the relation between such "vacuum pressure" and the velocity of the air through the tube, the instrument becomes an anemometer.

This described property of the Venturi tube was known to Venturi, and may be found detailed at length in the account of his experiments made in Modena about 1791. His own account of these experiments was published in Paris in 1797, under the title "Récherches expérimentales sur le Principe de la Communication latérale du Mouvement dans les Fluids."* But Venturi made or suggested no use of this property, and with him it was merely a curious feature in the working of his apparatus.

A pump made upon the principle of the Venturi tube may be found described in the *Zeitschrift des Ingenieur and Arch. Verein in Hanover*, 1873. It was exhibited by Nagel & Co., of Hamburg, in 1869. In this case the orifice into the throat of the Venturi tube was connected by means of a suction pipe with the water to be pumped.

And the following schedule gives the principal dimensions of the apparatus:

| | | |
|---|---|---|
| Fall acting through the Venturi tube | 8 | feet. |
| Height to which the water was raised | 8 | " |
| Diam. of suction pipe | 6 | inches. |
| Quantity raised | 1.75 | cub. ft. per sec. |
| Area at outlet of Venturi tube | 33.3 | square inches. |
| Area at smallest end | 10.1 | " |
| Area at throat, or narrowest passage for the "feed-water" | 3.6 | " |
| Area at narrowest section of suction water, at junction with the "feed-water" | 4.0 | " |

NOTE.—The last two measurements being for the case of best efficiency of the apparatus.

This pump replaced and did the same work as a 50 horse-power portable engine and centrifugal pump.

* See Tracts on Hydraulics, by Thomas Tredgold, second edition, London, 1836; or Nicholson's "Journal of Natural Philosophy," Vol. III, London, 1802, for English translations. Gilbert's "Annalen," Vol. II and Vol. III, contain a German translation.

Another use made of the Venturi tube was the "diffuser," as applied by Uriah A. Boyden to the Fourneyron turbine, its office having been to increase the discharge of the turbine, and consequently its efficiency; which apparatus has not, however, maintained itself in practice, owing to its cost being disproportionate to the gain accomplished with it.*

Then came the Bourdon Anemometer, above referred to, and now the instrument herein described, the Venturi Water Meter.†

It has been believed by some that the Romans made use, or rather abuse, of the properties of the Venturi tube, to take water unlawfully, the Roman water rights consisting, generally, of the right to insert into the public aqueducts and draw water through a calibered and officially stamped circular, tubular, bronze ajutage; which right, it has been thought, the over-selfishly disposed then tried to increase in value by the attachment, at the down stream end of the ajutage, of a Venturi tube.

Our principal source of knowledge on the subject is Sextus Julius Frontinus, who, to speak after the manner of the moderns, was superintendent, or sole water commissioner, of the water-works of Rome in A. D. 97, having been appointed in that year by the Emperor Nerva Augustus. My own copy of Frontinus is in German, and my knowledge of Latin is too rudimentary for service. But judging from the German translation, the phrase, *Calix devexus amplius rapit*, upon which the above recited impeachment, as well as tribute to the scientific skill, of the ancient Romans appears to have been founded, means quite a different thing, and assumes no knowledge of the properties of a Venturi tube in increasing the discharge. Its apparent meaning is, however, none the less interesting. As I translate the passage, with the context, it reads: "Ajutages inserted perpendicularly maintain a uniform discharge; if inclined and against the direction of the current, they discharge more (*calix devexus amplius rapit*); if inclined, but with the direction of the current, so that the water flows by, and the ajutage is less favorably placed for swallowing the water, it will drink without greed an insignificant quantity."

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* For a full account of the diffuser, see "Lowell Hydraulic Experiments." Also mentioned in later editions of "Weisbach's Mechanics."

† Bourdon's Anemometer may be found described in *Annales des Mines*, September and October, 1881; also in *Comptes Rendus*, January 30th, 1882, p. 229.

The above is also the meaning given the passage by Alfred Leger, *Travaux Publiques aux temps Romains*, p. 596.

The Roman water-takers were, however, undoubtedly "cute" enough to take all the advantages they could of the loose form of grant which depends for its limitations and definitions upon an officially stamped bronze ajutage, and it became necessary to pass a law prescribing that the pipe or conduit of each consumer should have the same diameter as his granted ajutage, for a space of not less than 50 lineal feet from the end of such ajutage. Venturi thought that his form of tube would increase the discharge, even if applied on the end of a pipe 50 feet long, though he says: "One ought to be suspicious of all hydraulic theories, of mine among the rest, so long as experiment has not confirmed them." Upon which Eytelwein (Gilbert's "Annalen," Vol. 7, p. 295) made a set of experiments to test this very point. He found that the effect of the Venturi ajutage, in the case of a discharge freely into the air, decreased as the tube, up-stream from it, lengthened. Discharging freely into the air, under 3 feet head, through a 1-inch tube 20 feet long, there was no apparent difference whether the Venturi ajutage was on, or off.

The experiments about to be detailed were made with two sizes of Venturi water meters, of precisely similar interior geometric dimensions; one inserted into a tube of about nine feet, the other into a tube of about one foot in diameter. In each case the other intended dimensions may be found from an examination of the proportional dimensions given in Plate XXXIII.

The throat, or the narrowest section of the whole apparatus, is a cylinder 1 high or long, and 3 in diameter. At the distance of 1 either way from the throat, are attached the frustrums of two cones, but the angles at which the cones would meet the cylinder are rounded off; in case of the up-stream cone, on a radius of 10.38; in case of the down-stream and longer cone (the Venturi mouth-piece), on a radius of 45.83. These figures are got from making the tangents of the rounded off portions in each case = 1.; the angles at the bases of the short and long cones being 79½ and 87½ degrees respectively. The cones are produced in each case until their diameter = 9.; making the lengths from throat to end of cone 17.09 and 69.80, respectively, and the length of the whole apparatus 87.89. These are the intended dimensions, in feet, of the larger apparatus experimented with October (5–8), and by dividing all these figures by 9., we get the intended dimensions in feet, of the smaller Venturi water meter, which was tested June (9–15), 1887.

Confining ourselves now to a consideration of this last named smaller apparatus, the meter itself is shown in Plate XXXIV. The throat or venturi, as it was named, is made of cast-iron, lined with brass, and comprises the central cylindrical and adjacent two curved portions of Plate XXXIII. Its total length was 0.563 feet. The brass lining was about ½ an inch thick, firmly set in its envelope of cast-iron and the joint between the two end faces, cut out in form of a dove-tailed circular slot, which was then filled with Babbitt metal to guard against a possible leakage of air through the joint.

Encircling the interior narrowest section is the air-chamber, which is connected with the interior by 4 accurately and carefully drilled holes, at right angles to the center line of the Venturi, and about ¼ inch in diameter each. The interior of the Venturi was carefully polished with emery dust after the holes were drilled, making the edges of the 4 holes, as finally left and used, perfectly square and sharp.

To measure accurately the area of the venturi, I had made a brass cylinder, which exactly fitted it, when both were of the same temperature. This cylinder I then measured, on 3 diameters, with a vernier caliper made by Darling, Brown & Sharpe, of Providence, R. I.

The averages in each case of three such measurements, when the plug had been standing all day in the air at a temperature of about 68 degrees, after a ½-hour's immersion in ice-water, and immediately after taking it out of water of a temperature of 160 degrees Fahr., were as follows:

| | | | |
|---|---|---|---|
| At 36 degrees Fahr. | 3.978 inches | | with no single measurement positive as to the final 0.001 inch. |
| 68 " " | 3.979 " | | |
| 160 " " | 3.980 " | | |

From which I computed the area, at about 60 degrees Fahr., to be 0.08634 square feet, and took this as constant in all the experiments.

The air-chamber is bored at the top to receive the suction pipe, to which may be attached any form of vacuum gauge.

In this, the first apparatus experimented with, there were several imperfections, which later experience pointed out. Some of these were corrected immediately, some were avoided for the first time in the construction of the 9-foot Venturi meter, and some remain suggestions for future use. To give this record the greatest utility it is capable of, it is necessary to point out these defects, or suggestions for improvement, and with that intent they are now enumerated.

*First.*—By inadvertence of the mechanics, the venturi was not set so as to bring the holes between the air-chamber and throat, exactly at the top and bottom, and on the horizontal diameter; and, at the time, this was not deemed of sufficient importance to justify the expense and delay of making the change and correction. These holes were 17 degrees out of their proper position, and this may, or it may not, have somewhat affected the results. The second apparatus had no such defect.

*Second.*—The tube leading directly from the air-chamber should have been made of glass, for reasons which will presently appear. This was so made in the second apparatus.

*Third.*—To gain an insight into the workings of the whole apparatus, the air-chamber should have been provided with pet-cocks, and these were inserted after the thirty-sixth experiment. They were also used in the second set of experiments.

*Fourth.*—The tube leading from the air-chamber should have had a pet-cock inserted into it, and leading into its interior, at a point higher up than the water ever stands in the tube. The absence of this adjunct spoiled the results of experiments (17–31), in the first set of experiments. It was used during the second, or October experiments. Changes in the apparatus not yet tried, will be spoken of later.

To either end of the central cast-iron member, the venturi, were attached two wooden cones, of the general interior dimensions already stated.

These were made of white pine staves, originally, or in the rough, about 2 inches thick, hooped with stout cast-iron hoops, carefully planed and scraped to smoothness inside, then soaked in water before using. The actual dimensions of the cones, measured after soaking in water, and again after the experiments, were as follows: giving the average of all the measurements taken; the differences between the first and second set being in no single measurement over 2 or 3 thousandths of a foot, and therefore insignificant.

| | | | | |
|---|---|---|---|---|
| Smaller cone, | length, | 1.677, | diameters, | .372 and .991 feet. |
| Larger cone, | " | 7.366, | " | .334 and .992 " |

For purposes of the experiments, the Venturi water meter thus formed of the two cones and the venturi, was inserted in line of a wooden tube, made, and treated before using, as just described for the case of the two cones. The up-stream length of tube had the following dimensions:

Length .....5.007; diameter at up-stream end, .990,
at down-stream end, .992.
Down-stream tube:
Length.....5.996; diameter..... .996, .998.

One foot from that end of these tubes which was joined to either of the two cones, each tube was bored to receive a piece of galvanized iron pipe, ¾-inch inside diameter. To bore these holes, a plug of soft wood was carefully fitted into the tube, under the place where the hole was to be bored. Then by using a center-bit auger, the hole could be cut through without roughing up the inside edges of the hole, and this hole left in proper shape for serving as the inside orifice of a piezometer.

The iron pipe spoken of, was screwed into this hole from the outside, but was not allowed to penetrate more than about half way into the thickness of the wooden stave, .162 feet thick, forming the tube at that point.

To feed the water to the up-stream tube, without loss of head at entrance, it was furnished with a cycloidal mouth-piece, likewise made of wooden staves, carefully smoothed and soaked in water as were the other members.

This mouth-piece had a diameter of 1.001 at the outlet, 2.50 feet at the inlet end, and was 1.17 feet long; its cycloidal generator would itself be generated by a point on a circle of 0.75 feet diameter.

The experiments were conducted in the wheel-pit of the testing flume of the Holyoke Water Power Company, a ground plan of which is shown in Plate XXXV. This is a building used by the company named for testing turbines, both for purposes of the water-measurements necessary in the conduct of its own affairs, and for the public. Its foundation masonry is first class and well grouted rubble, afterwards plastered with cement, and lined with brick laid in cement. The wheel-pit end-wall, built in this same manner, is absolutely water-tight under 20 feet head of water, and it is believed that all the other walls are equally firm and water-tight. The floor consists of matched 4-inch hemlock plank, spiked to timbers resting on rows of piles, and having 2.5-foot bearing, center to center, under the wheel-pit; 4-foot bearing under the tail-race. On this first flooring is spiked another, of 4-inch matched hard pine plank under the wheel-pit; of 2-inch matched white pine under the tail-race.

These statements will give a fair idea of the character of the structure which was used as a measuring tank in these experiments. For this purpose the brick walls were again plastered over with cement to give a smooth surface to measure from. The walls were then marked and divided off into rectangles by horizontal lines 1 foot apart, and by vertical lines 2 feet apart, and all dimensions carefully taken; while the floor was leveled on, both when the pit and tail-race were empty, and when full of water. I will not go into further details relating to the determination of the volume contained in the masonry tank between any two water-surfaces; nor into the determination of leakages, well known to be one of the most troublesome and laborious parts of the conduct of hydraulic investigations. Suffice it to say, that every thought of refinement of measurement and of computation was applied to the determination of volumes, while the accompanying leakages were measured at the beginning and ending of every experiment by noting the rate of rise or fall of the water surface in the tank. This water-surface being generally below the level of the adjacent lower level canal, the resultant leakage was sometimes in, sometimes out, and sometimes zero.

It was never over 0.11 cubic foot per second.

The heights of water in the tank were measured by two hook-gauges, having, together, a range of about 4 feet in height, and the area of the measuring tank was about 1 150 square feet.

The whole experimental apparatus is shown, in ground plan, in Plate XXXV; in elevation, in Plate XXXVI. The water entering through the gate $A$, passed through several temporary divisions put into the forebay $B$, to quiet it, then was fed to the tube and Venturi meter by the cycloidal mouth-piece above referred to. It was discharged into the tank $C$, whence it flowed either into the box $D$, leading through the waste-pipe $E$, to a by-pass $F$, or else, on swinging back the spout $G$, it was discharged into the measuring tank below. $P$ and $P_1$ are piezometers, the difference of their readings indicating the head acting on the whole meter; or loss of head, caused by it. $S$ is the suction end of the vacuum tube $V$, in these experiments dipped into a tub of water, and the whole tube being 34.5 feet long, was long enough to measure a perfect vacuum, had such a thing been attainable.

This last named part of the tube was of course made of glass, in 5 lengths and with rubber-tube joints. Great care was taken to make all the joints air-tight, and by means of wrapping them with telegrapher's

rubber-tape, and the use of rubber cement, this was finally, it was believed, successfully accomplished.

Our positive knowledge on the subject is, that in the experiments following experiment No. 7, no more air-bubbles could be detected rising through the water within the vacuum tube. This is a very delicate test for leaks in all joints below this water level; in case of the other joints, reliance had to be placed on careful work equal in quality to that used in the tested joints.

A drip-box *H* caught the leakage of the spout *G*, while the water was wasting, and discharged this leakage into the waste-pipe *E* by means of the pipe *I*. *J* is a waste-valve leading to the by-pass *F*, which helped to regulate the height of water in the forebay *B*.

At the moment of one assistant opening the swing-spout *G* to discharge the water into the measuring tank, another assistant shut off the pipe *I* by means of an ordinary pipe valve next the drip-box *H*, and he opened it again, when the swing-spout was swung back into contact with the tank *C*. The contents of the drip-box *H*, which wasted into the waste-pipe *E* at the close of each experiment, when they should have gone into the measuring tank, were added to the volume found in the measuring tank.

The times when the swing-spout was opened and when shut, were taken by the writer, with a stop-watch, reading to ¼ seconds.

The practice of the first three or four experiments sufficed to get this time, as near as he could judge, exactly right. The swing-spout was handled so easily, by means of the long lever attached, that its motion was very quick and positive. Inside of the tank *C*, and in front of the tube discharging into it, was a sliding gate, by means of which the discharge of the tube, and of the Venturi meter, could be regulated.

It will be noted that the discharge took place under water; or, as it is generally expressed, the whole apparatus was submerged. At a later stage of the experiments, when it became desirable to reduce the amount of submergence more than was permitted by the position of the swing-spout, several experiments were made with the discharge escaping from the tank *C*, through a series of holes bored into it; this discharge being measured, by comparison with other discharges of the whole system of tube, Venturi meter and tube, when acting under the same total heads.

This series consists of experiments (70–76).

Another set were made at a time when there was some doubt as to

the reliability of the whole apparatus, and consisted merely of roughly measuring the total heads and the corresponding indications of the vacuum gauge, and running this latter, by means of the gate in the tank *C*, up and down its extreme range several times as quickly as possible, and without regard to the accuracy obtainable from the average reading of many observations taken under the same circumstances. This series consists of experiments (77–154), and in it likewise the actual discharge was not measured, and was obtained as in the series (70–76). All the data upon which it is founded are mere single readings of oscillating columns of water, taken for the purpose and in the manner stated.

Still another odd set of experiments is the series (56–60). To thoroughly explain this set it may be best to speak now of the actual operations of the Venturi meter as applied to a water-pipe in ordinary service.

If we suppose the water in the pipe to be still, the height of water in a piezometer placed just up-stream from the meter, and in the one formed by the suction pipe which leads out of the Venturi air-chamber, will be on the same level. When the water begins to flow through the Venturi it will cause the piezometric column leading out of the Venturi to fall below the straight line, joining the surfaces of the water in the two piezometric tubes placed, the one just up-stream, and the other just down-stream from the Venturi meter; this straight line being the best obtainable reference line at the time the experiments were being conducted.

This increment of fall I have called the "depression" at the Venturi. And in the series (56–60) this depression was seen through a galvanized iron pipe by "the eye of faith" alone. Its value was taken from the values found for such depression in other experiments, when the water was passing through the Venturi with similar velocities, and the degree of submergence was such that the depression could be measured in form of a vacuum and by the vacuum gauge. In the subsequent set, the October experiments, the suction tube referred to was of glass, and the depression could be directly measured.

To resume a description of the action of the whole apparatus: As the depression increases, there comes a time, when the water level in the suction tube will fall to the level of the top of the air-chamber, then fall still further, until finally it touches or blends itself with the sur-

face of the stream of water spouting through the Venturi. The moment the tendency to a depression tends to depress the water-column in the suction tube still further, a true sucking action commences. So long as the Venturi end of the suction pipe acts as a piezometer, it is necessary that this pipe be connected with the outer air by means of the pet-cock above spoken of, to have its indications reliable; as otherwise the air contained in the pipe between the Venturi and the tub of water, or mercury, at the lower end of the other leg of the suction pipe, may become compressed or rarified by the action of the water backing up, or of the Venturi exhausting air, and thus cause the piezometric readings to be in error. As soon as a true sucking action commences, however, the pet-cock must of course be closed. As stated, the apparatus used in the June tests had no such pet-cock, which defect rendered experiments (17–31) worthless.

The complete records of the experiments are too voluminous to be reproduced in print. Before giving the results in tabular, digested form, therefore, some of the characteristics of the observations taken will be described.

The upper head-gauge, column 5 of the table, was liable to have large bubbles of air rise up through it when the water in the forebay fell too low, or was too much churned up by its discharge through the head-gate. When this occurred, the experiments were, as a rule, suspended, and the trouble was remedied by raising the water level in the forebay, or by causing the water to flow through longer channels before reaching the cycloidal mouth-piece. The water in the forebay could not be kept in sight, and during some of the experiments with a low stage of water (70–76) an eddy may have formed above and next the mouth-piece, and carried air into and through the meter. I regard it probable that the divergence of experiments (70–76) from the mean was due to this cause. But during the experiments the indications of head-gauge No. 1 as to air-bubbles were regarded as conclusive, with respect to the presence of air, or of as little air as was practically attainable, in the water carried by the Venturi meter. When air-bubbles were seen in this gauge-tube, the flow of water to the mouth-piece was ameliorated; and when none were seen, it was supposed that there was no need for the amelioration referred to. At first, readings were taken on this and on head-guage No. 2, every minute; but after the eleventh experiment they were taken at least every half-minute, and sometimes as many as four per minute, during the duration of the experiment.

The oscillations of the water column in head-gauge No. 1 were not

large, being in some experiments as little as 0.02 in the course of the experiment, and seldom, if ever, touching 0.10 as their extreme range.

The down stream head-gauge, column 6 of the table, was more troublesome as to air-bubbles coming up in it, and in its range of oscillations. Air-bubbles could probably have been made to pass by unnoticed, by tapping the piezometer into the bottom of the 1-foot tube; but the original way of tapping it into the top of the tube was adhered to, for the very purpose of being thereby able to judge, somewhat, of the amount of air carried through the meter. Low velocities caused least oscillations, from .05 to .10; high velocities caused greater oscillations in the piezometric column, ranging as high as 0.30, in some cases 0.35, during the duration of a single experiment, but without any effort being made to register maxima and minima. The amount of air carried through the meter was naturally greatest during the experiments with high velocities. It was also governed, no doubt, to some extent by the degree of submergence of the whole apparatus. It was never allowed to be great enough to cast palpable discredit on the indications of the gauge No. 2, having regard to such indications consisting of the average of the many readings (sometimes forty or more) noted down during the course of a single experiment.

The vacuum gauge, column 9 of the table, was supplied with water of very nearly the same temperature as that of the water passing the meter. The gauge alongside of the water-barometer tube was movable, and had for its zero the point of a hook-gauge dipping into the tub of water that supplied the water for the barometric column. The point of this hook-gauge, or zero of the gauge, could thus be constantly kept at the level of the water in the tub.

During extremes of velocity, the oscillations of the water column were least; for velocities of 15. to 35. feet per second through the venturi they were greater. Two methods of reading this gauge were used. In the one, the observer made his record once a minute, mentally noting, before writing down the observation, what was the average of the oscillations seen. This method gave an extreme range of .10 or .11 during the course of any single experiment. In the other method, the observer noted down heights seen, as fast as he could write, so as to catch the very extremes of all the oscillations. This method, gave sometimes as much as two feet of oscillation during the course of a single experiment. The use of a mercury column instead of a water column would naturally have limited these ranges of oscillation to about $\frac{1}{14}$ of their value as found. But it was deemed best to use the water column for purposes of an accurate representation of the forces at work in the apparatus. On the other hand, the air-chamber was intended to average or to quiet the action of these forces, as they might act through a single orifice bored into the venturi, and directly connected with a piezometric tube or with the suction pipe of a vacuum barometer. At date of this

writing, I regard the use of some form of air-chamber as essential to the good working of a venturi meter.

When the vacuum column was broken by opening either of the pet-cocks which were above the water level in the air-chamber, the water column, previously supported by the existence of the partial vacuum, would drop instantly, then perhaps oscillate, with a downward tendency in the oscillations.

Its fall could, as a rule, be as instantly arrested, by closing the pet-cock orifice with the finger; and sometimes a number of taps with the finger would be telegraphically repeated by apparently synchronous movements of the water column.

Before presenting the tabular results of the experiments, I also present some remarks as to the methods of computation which yielded these results. The first six columns of the table contain data, and with what has been said above, will need no further explanation.

Column 7 is a mere subtraction of Column 6 from Column 5.

Column 8 will be as readily understood.

Column 9 contains data, being the length of the water column held up in the vacuum gauge, and measured as already described.

Column 10 shows the working of the 3 pet-cocks that were tapped into the air-chamber, one on top, one on line with a horizontal diameter, and one directly at the lowest point of the air-chamber.

Column 11 is the "head on the venturi," or $H_v$, being the difference of level between the water at, and above the venturi, as indicated by the head-gauge No. 1 and by the vacuum gauge. The elevation of the top of the inside of the venturi was 84.704, and all measured vacuum heights must be substracted from this quantity to get the constructive elevation of the water, or locus of the hydraulic gradient, at the venturi. The difference between this locus, or elevation of the hydraulic gradient, at head-gauge No. 1 and at the venturi is the "head on the venturi," in the computations as made and recorded in the table.

Column 12 contains the co-efficient belonging to the ordinary computation of discharge through an orifice, when the discharge is as found during the experiment, the orifice is the venturi, and the head is $H_v$.

Column 13 is the locus of a point at the venturi in a straight line, connecting the points in the hydraulic gradient found at head-gauges Nos. 1 and 2.

Column 14 is the difference between Column 13 and 84.704, plus the measured vacuum heights. It indicates how much the hydraulic gradient was depressed at the venturi below the point given in Column 13. From Columns 13, 14 and 5 was found the $H_v$ to be used in experiments (56-60). With a more perfect experimental apparatus, this and other roundabout methods of interpolation used in computing the tabular results could, of course, have been avoided. But with the data at hand, it was deemed better to utilize all there were, in the best way possible rather than reject any.

Column 15 indicates the characters used in plotting the experiments.

Column 16 is $H_v$ plus the height due the velocity of approach in the one-foot tube; and is called $H_v'$.

Column 17 is the co-efficient corresponding to $H_v'$ when used similarly to $H_v$ of Column 11.

A star affixed to a number in the table indicates some form of interpolation.

Experiment No. 66 is worthless on account of inability to keep the head steady in the forebay. Experiments 14 and 15 are unreliable on account of too much air in the down-stream head-gauge and passing the meter. And I hold experiments 70–76 to be unreliable, as already stated.

It is much to be desired that the whole series be repeated with the more perfect means for observation, which the experience of this, the first series, suggested.

The October set of experiments were made with a Venturi meter built up inside of the 9-foot trunk that feeds the testing flume. This trunk is of the sort usually built in Holyoke, of boiler iron, to serve as a penstock of mills.

Every alternate ring, or section of about 4½ feet in length, is a spigot-piece, at both ends, and the others are bell-pieces. Each ring is formed of three plates, lapped and riveted, the bell and spigot-joints also being riveted. Nominally 9 feet in diameter, very careful measurement on 15 diameters, at the 35 narrowest sections of a length of about 150 feet, gave the average area, rivet-heads subtracted, 57.823 square feet. The area of the "average shape" was 57.742 square feet.

This "average shape" was very nearly a true ellipse, having 8.70 and 8.93 for its minor and major axes, the minor axis being the vertical one.

Commencing at the upper level canal, comes the usual rack to keep out floating substances; then a head-gate, in these experiments wide open, and of no influence; then a piece of trunk, about 7 feet in diameter and 22 feet long; then a conical piece, 8 feet long, to expand from the 7-foot to the 9-foot trunk; then the 9-foot trunk, in a straight line with the two pieces already named, 224 feet, then curving on a quarter circle of 40 feet radius of center line, and a straight length 9 feet long, to the vestibule of the testing flume, as indicated in Plate XXXV.

Head-gauge No. 1 was situated 73.92 feet down-stream from the inside of the rack, the inside of the rack being 35.5 feet up-stream

from the up-stream end of the 9-foot trunk. From head-gauge No. 1 to the beginning of the up-stream cone was 36.81 feet; thence to center of the venturi, 16.10 feet; thence to end of lower cone, 69.58 feet; thence to head-guage No. 2, 30.39 feet.

This trunk has an inclination down-stream of 1.577 in 100 feet, as measured from the interior surfaces of the orifices leading to head-gauges Nos. 1 and 2. All the structures placed in it, hereafter to be described, were set in planes at right angles to the center line, or so as to have the center line for their geometric axis. The inside of the top of the venturi was on grade 90.909. The water-level in the upper canal at this point is about 99.90; that of the lower-level canal about 79.90.

As above stated, the general dimensions of the meter were intended as given in Plate XXXIII. The venturi is shown in Plate XXXVII, and was made, as in the 1-foot meter, of cast-iron, lined with brass; differing from the 1-foot venturi, however, in having eight separate air-chambers, one for each ¼-inch hole leading out of the venturi.

These several air-chambers had each a suction-pipe attached, and a pet cock, as shown in Plate XXXVII. The several suction-pipes had each a stop-valve, and were then assembled into the main suction-pipe, which led to the vacuum-gauge or water-barometer. By means of this arrangement, any one, all, or combination of several air-chambers could be connected with the vacuum gauge to the exclusion of the others.

Plate XXXVIII shows the whole meter in longitudinal section. The iron and brass venturi was, in this case, as will be seen, a true cylinder, about 3 feet in diameter and only 1 foot long. Its area was 7.07425 square feet, as determined from a measurement, with a vernier slide-rod, of 12 diameters; 4 at each end and at the center of the cylinder.

At either end of the venturi was a wooden connecting-piece, made to flare outwardly in the curves shown in Plate XXXIII, and built up of staves, hooped with strong cast-iron frames, turned and smoothed in a lathe and soaked in water before setting. At either end of the central portion thus formed came the two cones, consisting of planed pine strips nailed to circular frames or hoops, set inside the 9-foot trunk. This construction of the cones did not leave them so smooth, at all the joints and butts, as was the case with the 1-foot cones, of course; the whole surface was, however, much smoother than the interior of the iron trunk at either end. Two water-tight bulk-heads set in the trunk at either

TABLE No. 1

# HERSCHEL ON VENTURI WATER METER.

## VENTURI EXPERIMENTS, JUNE, 1887.



| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | | 16 | 17 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Head Gauges. | | | | | | | | | | Designation of Expts. | | | |
| No. of experiment. | Duration of experiments. Seconds | Total volume of water passed Cu. ft. | Velocity through venturi. Feet per second. | Upper. | Lower | $H$. Total head Feet | Co-efficient. Velocity. $= \sqrt{2g\,H}$ | Vacuum gauge. Height of water column. Feet. | Working of pet cocks. | $H_0$. Head on venturi. Feet. | Co-efficient. Velocity. $= \sqrt{2g\,H_0}$ | Hydraulic gradient at venturi. | Depression below hydraulic gradient at venturi. | $H_0$. Curve. a | Depression curve b | $H$. curve. c | Theoretical $H$ c. | Corresponding co-efficient. |
| 1 | 987.25 | 4236.0 | 49.70 | 99.060 | 88.551 | 10.718 | 1.893 | 24.509 | Assume air in all. | 38.874 | 0.994 | 96.362 | ..... | ○ | .. | ○ | 39.355 | 0.988 |
| 2 | 592.75 | 2289.4 | 49.62 | 99.376 | 88.329 | 11.096 | 1.861 | 24.512 | " | 39.184 | 0.988 | 96.583 | ..... | .. | .. | .. | 39.661 | 0.982 |
| 3 | 992.50 | 4102.4 | 49.88 | 99.492 | 88.565 | 11.137 | 1.872 | 24.531 | " | 39.229 | 0.993 | 96.614 | ..... | | | | 39.714 | 0.987 |
| 4 | 778.00 | 3312.0 | 49.80 | 99.360 | 88.359 | 11.194 | 1.869 | 24.599 | " | 39.288 | 0.991 | 96.606 | ..... | | | | 39.771 | 0.985 |
| 5 | 682.75 | 2793.3 | 45.86 | 94.173 | 88.300 | 5.874 | 2.359 | 24.685 | " | 33.154 | 0.993 | 92.690 | ..... | | | | 33.564 | 0.987 |
| 6 | 483.75 | 1917.1 | 45.00 | 94.184 | 88.340 | 5.844 | 2.367 | 24.855 | " | 33.135 | 0.994 | 92.698 | ..... | | | | 33.545 | 0.988 |
| 7 | 587.50 | 2208.3 | 44.53 | 94.258 | 89.140 | 4.854 | 2.464 | 19.704 | " | 29.234 | 1.004 | 93.028 | ..... | | | | 29.624 | 0.997 |
| 8 | 593.75 | 2221.2 | 43.33 | 94.238 | 89.400 | 4.838 | 2.456 | 19.339 | " | 28.873 | 1.005 | 93.016 | ..... | | | | 29.239 | 0.999 |
| 9 | 600.00 | 2067.7 | 39.72 | 94.744 | 90.560 | 3.784 | 2.546 | 14.565 | " | 24.205 | 1.007 | 93.388 | ..... | | | | 24.513 | 1.000 |
| 10 | 679.00 | 1948.4 | 36.98 | 94.332 | 90.562 | 3.770 | 2.501 | 14.672 | " | 24.300 | 0.986 | 93.380 | ..... | | | | 24.595 | 0.980 |
| 11 | 251.00 | 846.7 | 32.07 | 94.326 | 90.557 | 3.758 | 2.504 | 14.448 | " | 24.079 | 0.995 | 93.361 | ..... | | | | 24.276 | 0.989 |
| 12 | 601.75 | 1461.9 | 33.78 | 94.437 | 91.643 | 2.794 | 2.522 | 8.251 | " | 17.974 | 0.992 | 93.724 | ..... | | | | 18.166 | 0.986 |
| 13 | 767.25 | 2242.7 | 33.85 | 94.417 | 91.003 | 2.814 | 2.516 | 8.307 | " | 18.020 | 0.994 | 93.706 | ..... | | | | 18.243 | 0.988 |
| 14 | 667.50 | 1503.2 | 28.33 | 94.669 | 93.040 | 1.799 | 2.508 | 7.215 | " | 14.120 | 0.993 | 91.104 | ..... | | | | 14.277 | 0.988 |
| 15 | 469.25 | 1159.7 | 28.62 | 94.699 | 93.585 | 2.014 | 2.515 | 5.704 | " | 12.599 | 1.003 | 91.002 | ..... | | | | 12.780 | 1.002 |
| 16 | 491.75 | 1195.3 | 28.15 | 91.605 | 89.049 | 1.936 | 2.510 | 5.787 | " | 11.080 | 1.014 | 91.111 | ..... | ○ | | | 12.134 | 1.008 |
| 17 | 515.00 | 955.4 | 21.49 | 91.350 | 90.310 | 1.176 | 2.170 | 3.228 | ..... | ..... | ..... | 91.389 | ..... | | | | ..... | |
| 18 | 225.75 | 417.9 | 21.44 | 91.700 | 90.525 | 1.175 | 2.166 | 3.056 | ..... | ..... | ..... | 91.403 | ..... | | | | ..... | |
| 19 | 554.00 | 100.6 | 2.85 | 91.876 | 91.846 | 0.030 | 2.051 | 0.951 | ..... | ..... | ..... | 91.806 | ..... | | | | ..... | |
| 20 | 583.00 | 770.4 | 16.42 | 91.950 | 91.356 | 0.594 | 2.495 | 9.324 | ..... | ..... | ..... | 91.800 | ..... | | | | ..... | |
| 21 | 218.75 | 502.3 | 15.76 | 91.969 | 91.553 | 0.616 | 2.478 | 9.442 | ..... | ..... | ..... | 91.813 | ..... | | | | ..... | ..... |
| 22 | 147.50 | 669.6 | 17.33 | 91.949 | 91.162 | 0.787 | 2.436 | 9.564 | ..... | ..... | ..... | 91.750 | ..... | | | | ..... | |
| 23 | 591.75 | 893.7 | 17.49 | 91.952 | 91.143 | 0.789 | 2.453 | 9.538 | ..... | ..... | ..... | 91.753 | ..... | | | | ..... | |
| 24 | 498.50 | 654.0 | 15.19 | 91.966 | 91.387 | 0.578 | 2.492 | 9.052 | ..... | ..... | ..... | 91.819 | ..... | | | | ..... | |
| 25 | 465.50 | 958.2 | 24.77 | 91.153 | 90.045 | 1.338 | 2.562 | 3.332 | ..... | ..... | ..... | 91.815 | ..... | | | | ..... | |
| 26 | 488.50 | 1039.6 | 24.41 | 92.149 | 90.844 | 1.315 | 2.554 | 3.166 | ..... | ..... | ..... | 91.817 | ..... | | | | ..... | |
| 27 | 406.75 | 737.5 | 17.20 | 92.243 | 91.652 | 0.591 | 2.520 | 1.538 | ..... | ..... | ..... | 92.094 | ..... | | | | ..... | |
| 28 | 590.75 | 176.2 | 3.64 | 92.548 | 92.499 | 0.049 | 2.050 | 0.427 | ..... | ..... | ..... | 92.536 | ..... | | | | ..... | |
| 29 | 565.25 | 187.7 | 3.29 | 92.544 | 92.496 | 0.048 | 1.872 | 0.400 | ..... | ..... | ..... | 92.532 | ..... | | | | ..... | ..... |
| 30 | 490.25 | 742.8 | 16.44 | 92.450 | 91.744 | 0.662 | 2.484 | 1.399 | ..... | ..... | ..... | 91.901 | ..... | | | | ..... | |
| 31 | 356.50 | 471.7 | 15.32 | 91.626 | 91.140 | 0.586 | 2.496 | 1.801 | ..... | ..... | ..... | 91.598 | ..... | | | | ..... | |
| 32 | 351.50 | 1650.9 | 51.10 | 99.293 | 88.367 | 10.836 | 1.936 | 24.828 | Assume air in all. | 39.327 | 1.016 | 96.466 | ..... | ○ | | | 39.836 | 1.010 |
| 33 | 378.50 | 1650.5 | 50.61 | 99.135 | 88.478 | 10.777 | 1.918 | 24.681 | " | 39.132 | 1.007 | 96.433 | ..... | | | | 39.629 | 1.000 |
| 34 | 403.25 | 1752.4 | 50.33 | 99.222 | 91.028 | 8.199 | 2.192 | 24.116 | " | 38.634 | 1.010 | 95.151 | ..... | | | | 39.127 | 1.003 |
| 35 | 424.75 | 1715.6 | 46.78 | 99.240 | 93.671 | 5.569 | 2.472 | 19.358 | " | 33.894 | 1.002 | 97.833 | ..... | | | | 34.320 | 0.996 |
| 36 | 426.75 | 1469.6 | 39.80 | 99.444 | 95.665 | 3.779 | 2.558 | 10.018 | " | 24.758 | 0.997 | 98.489 | ..... | ○ | | | 25.067 | 0.993 |
| 37 | 375.75 | 818.8 | 25.24 | 94.200 | 92.600 | 1.600 | 2.488 | 0.520 | Top draws air. | 10.016 | 0.994 | 93.796 | 9.642 | ○ | ○ | | 10.140 | 0.988 |
| 38 | 535.00 | 986.8 | 21.36 | 91.650 | 90.600 | 1.150 | 2.484 | 0.275 | T. & M. air. B water | 7.321 | 0.984 | 91.350 | 7.034 | | | | 7.410 | 0.979 |
| 39 | 486.75 | 936.5 | 22.28 | 92.350 | 91.100 | 1.250 | 2.485 | 0.556 | " | 8.202 | 0.970 | 92.034 | 7.888 | | | | 8.298 | 0.965 |
| 40 | 416.75 | 1177.8 | 26.30 | 93.150 | 93.400 | 1.052 | 2.551 | 0.564 | " | 11.006 | 0.988 | 94.134 | 10.592 | | | | 11.144 | 0.982 |
| 41 | 502.25 | 1196.5 | 27.59 | 95.150 | 93.350 | 1.800 | 2.564 | 1.643 | All air | 12.089 | 0.989 | 94.695 | 11.634 | | | | 12.247 | 0.983 |
| 42 | 441.00 | 1098.3 | 28.84 | 95.500 | 93.500 | 2.000 | 2.543 | 2.511 | " | 13.397 | 0.988 | 94.955 | 12.804 | | | | 13.469 | 0.980 |
| 43 | 428.75 | 1102.3 | 29.78 | 95.494 | 93.350 | 2.144 | 2.560 | 3.463 | " | 14.213 | 0.985 | 94.924 | 13.682 | | | | 14.385 | 0.979 |
| 44 | 312.00 | 885.6 | 32.88 | 95.420 | 92.900 | 2.520 | 2.582 | 6.286 | " | 17.002 | 0.994 | 94.783 | 16.365 | | | | 17.212 | 0.988 |
| 45 | 361.25 | 1109.2 | 35.66 | 95.360 | 92.350 | 3.030 | 2.547 | 9.342 | " | 20.018 | 0.991 | 94.645 | 19.253 | | | | 20.284 | 0.985 |
| 46 | 347.50 | 842.8 | 41.39 | 95.300 | 91.307 | 3.993 | 2.583 | 15.112 | " | 25.708 | 1.018 | 94.291 | 24.692 | | | | 26.041 | 1.011 |
| 47 | 231.25 | 940.0 | 47.98 | 95.600 | 90.000 | 5.600 | 2.481 | 24.045 | " | 34.941 | 0.983 | 94.146 | 33.827 | | | | 35.373 | 0.977 |
| 48 | 252.25 | 1071.4 | 49.19 | 95.600 | 88.200 | 7.400 | 2.255 | 28.477 | " | 38.373 | 1.017 | 93.731 | 34.594 | | | | 36.844 | 1.011 |
| 49 | 409.50 | 1158.9 | 32.78 | 95.443 | 93.300 | 2.443 | 2.015 | 6.697 | " | 16.836 | 0.996 | 94.826 | 18.219 | | | | 17.045 | 0.990 |
| 50 | 433.00 | 1091.4 | 29.19 | 95.500 | 93.600 | 1.900 | 2.641 | 2.339 | " | 13.035 | 1.008 | 95.020 | 12.555 | | | | 13.201 | 1.002 |
| 51 | 252.75 | 606.6 | 27.80 | 95.500 | 93.750 | 1.750 | 2.620 | 1.038 | " | 11.834 | 1.008 | 95.058 | 11.392 | | ○ | | 11.984 | 1.001 |
| 52 | 464.00 | 1052.8 | 26.28 | 95.500 | 93.900 | 1.600 | 2.590 | 0.246 | T. air. M air & water. B water | 11.042 | 0.985 | 95.096 | 10.638 | | | | 11.176 | 0.980 |
| 53 | 602.00 | 1051.9 | 18.21 | 88.896 | 88.053 | 0.843 | 2.473 | 1.171 | All air. T. & M air. | 5.363 | 0.980 | 88.883 | 5.160 | | | | 5.428 | 0.975 |
| 54 | 557.50 | 814.2 | 16.92 | 88.922 | 88.191 | 0.731 | 2.467 | 0.394 | B. water. T. air | 4.612 | 0.982 | 88.737 | 4.427 | | | | 4.667 | 0.975 |
| 55 | 623.50 | 904.7 | 16.75 | 88.912 | 88.219 | 0.693 | 2.509 | 0.211 | M & B. water. | 4.410 | 0.994 | 88.737 | 4.244 | ○ | ○ | | 4.474 | 0.988 |
| 56 | 505.25 | 616.7 | 14.14 | 88.939 | 88.459 | 0.510 | 2.493 | ..... | Water level at the venturi above the air chamber. | 3.20 | 0.985 | 88.833 | 3.07* | | | | 3.24 | 0.979 |
| 57 | 540.00 | 398.9 | 8.56 | 88.960 | 88.760 | 0.200 | 2.385 | ..... | | 1.21 | 0.970 | 88.902 | 1.16* | | | | 1.22 | 0.966 |
| 58 | 574.75 | 217.5 | 4.38 | 88.940 | 88.890 | 0.050 | 2.444 | ..... | | 0.36 | 0.941 | 88.927 | 0.35* | | | | 0.36 | 0.941 |
| 59 | 754.50 | 87.6 | 1.35 | 88.971 | 88.961 | 0.010 | 1.677 | ..... | | 0.05 | 0.750 | 88.968 | 0.05* | | | | 0.05 | 0.750 |
| 60 | 748.25 | 144.0 | 2.23 | 88.971 | 88.952 | 0.019 | 2.016 | ..... | | 0.10 | 0.879 | 88.966 | 0.10* | | ○ | ○ | 0.10 | 0.879 |
| 61 | ..... | .. | 16.00* | 88.850 | 88.050 | 0.800 | ..... | 1.005 | All air. | 5.151 | 0.989 | 88.648 | 4.949 | ⊗ | ○ | | 5.214 | 0.983 |
| 62 | ..... | ..... | 12.60* | 88.850 | 88.080 | 0.770 | ..... | 0.954 | T. & M. air. | 5.100 | 0.972 | 88.655 | 4.905 | | | | 5.190 | 0.966 |
| 63 | ..... | ..... | 17.12* | 88.850 | 88.120 | 0.730 | ..... | 0.863 | B. water | 4.709 | 0.984 | 88.665 | 4.524 | | | | 4.766 | 0.978 |
| 64 | ..... | ..... | 16.60* | 88.860 | 88.170 | 0.690 | ..... | 0.283 | " | 4.430 | 0.982 | 88.665 | 4.264 | | | | 4.493 | 0.975 |
| 65 | ..... | ..... | 16.05* | 88.865 | 88.220 | 0.645 | ..... | 0.019 | T. water. M. & B water. | 4.180 | 0.979 | 88.702 | 4.017 | | | | 4.230 | 0.973 |
| 66 | ..... | ..... | 12.00* | 88.350 | 87.976 | 0.374 | ..... | 0.008 | T air M. & B. water. | 3.654 | 0.783 | 88.255 | 3.539 | | | | 3.582 | 0.780 |
| 67 | ..... | ..... | 15.60* | 88.616 | 88.010 | 0.606 | ..... | 0.234 | The venturi | 4.156 | 0.948 | 88.463 | 4.003 | | | | 4.220 | 0.941 |
| 68 | ..... | ..... | 19.88* | 89.030 | 88.070 | 0.960 | ..... | 1.768 | All air. | 6.604 | 1.002 | 88.787 | 5.851 | | | | 5.171 | 0.995 |
| 69 | ..... | ..... | 18.72* | 88.915 | 88.053 | 0.862 | ..... | 1.332 | " | 5.643 | 0.991 | 88.697 | 5.325 | | | | 5.612 | 0.985 |
| 70 | ..... | ..... | 14.16* | 88.806 | 88.300 | 0.506 | ..... | 0.834 | " | 2.936 | 1.028 | 88.678 | 2.808 | | | | 2.975 | 1.019 |
| 71 | ..... | ..... | 16.20* | 89.426 | 88.767 | 0.659 | ..... | 2.094 | " | 3.805 | 1.035 | 88.769 | 3.639 | | | | 3.857 | 1.029 |
| 72 | ..... | ..... | 12.90* | 88.432 | 88.039 | 0.393 | ..... | 0.534 | " | 2.262 | 1.020 | 88.333 | 2.168 | | | | 2.292 | 1.013 |
| 73 | ..... | ..... | 11.25* | 88.484 | 86.162 | 0.334 | ..... | 0.160 | T. air M. & B. water. | 1.940 | 1.007 | 86.409 | 1.856 | | | | 1.965 | 1.001 |
| 74 | ..... | ..... | 10.40* | 88.309 | 86.117 | 0.282 | ..... | 0.015 | " | 1.710 | 0.992 | 86.328 | 1.639 | | | | 1.731 | 0.986 |
| 75 | ..... | ..... | 8.35* | 83.892 | 83.699 | 0.193 | ..... | 0.104 | " | 1.292 | 0.916 | 83.843 | 1.243 | | | | 1.305 | 0.911 |
| 76 | ..... | ..... | 9.73* | 86.006 | 85.784 | 0.252 | ..... | 0.227 | T. & M. air. B. water. | 1.529 | 0.981 | 85.942 | 1.465 | ⊗ | ○ | | 1.547 | 0.975 |
| 77 | ..... | ..... | 22.38* | 94.28 | 94.08 | 1.20 | ..... | 0.00 | ..... | ..... | ..... | ..... | ..... | | | | ..... | |
| 78 | ..... | ..... | 26.97* | 94.25 | 92.55 | 1.70 | ..... | 0.51 | Assume air in all. | 10.00 | 1.060 | ..... | ..... | | | | ..... | |
| 79 | ..... | ..... | 28.57* | 94.25 | 92.35 | 1.90 | ..... | 2.15 | " | 11.70 | 1.041 | ..... | ..... | | | | ..... | ..... |
| 80 | ..... | ..... | 32.70* | 94.102 | 91.60 | 2.50 | ..... | 4.07 | " | 13.47 | 1.111 | ..... | ..... | | | | ..... | ..... |
| 81 | ..... | ..... | 32.70* | 94.10 | 91.70 | 2.50 | ..... | 6.18 | " | 15.58 | 1.033 | ..... | ..... | | | | ..... | ..... |
| 82 | ..... | ..... | 34.34* | 94.10 | 91.40 | 2.70 | ..... | 8.10 | " | 17.00 | 1.031 | ..... | ..... | | | | ..... | ..... |
| 83 | ..... | ..... | 37.33* | 94.00 | 90.70 | 3.30 | ..... | 10.10 | " | 19.40 | 1.057 | ..... | ..... | | | | ..... | ..... |
| 84 | ..... | ..... | 37.88* | 93.90 | 90.60 | 3.40 | ..... | 13.00 | " | 22.20 | 1.002 | ..... | ..... | | | | ..... | ..... |
| 85 | ..... | ..... | 40.42* | 93.90 | 90.30 | 3.60 | ..... | 14.80 | " | 23.90 | 1.031 | ..... | ..... | | | | ..... | ..... |
| 86 | ..... | ..... | 41.80* | 93.80 | 89.60 | 4.20 | ..... | 17.60 | " | 26.70 | 1.069 | ..... | ..... | | | | ..... | ..... |
| 87 | ..... | ..... | 42.61* | 93.70 | 89.30 | 4.40 | ..... | 19.00 | " | 28.00 | 1.094 | ..... | ..... | | | | ..... | ..... |
| 88 | ..... | ..... | 44.80* | 93.65 | 88.65 | 5.00 | ..... | 21.20 | " | 30.15 | 1.017 | ..... | ..... | | | | ..... | ..... |
| 89 | ..... | ..... | 45.44* | 93.65 | 88.45 | 5.20 | ..... | 22.85 | " | 31.80 | 1.005 | ..... | ..... | | | | ..... | ..... |
| 90 | ..... | ..... | 46.03* | 93.60 | 88.20 | 5.40 | ..... | 24.25 | " | 33.15 | 0.987 | ..... | ..... | | | | ..... | ..... |
| 91 | ..... | ..... | 45.12* | 93.60 | 88.50 | 5.10 | ..... | 22.50 | " | 31.40 | 1.004 | ..... | ..... | | | | ..... | ..... |
| 92 | ..... | ..... | 44.76* | 93.85 | 88.95 | 4.70 | ..... | 20.60 | " | 29.55 | 1.004 | ..... | ..... | | | | ..... | ..... |
| 93 | ..... | ..... | 41.43* | 93.75 | 89.65 | 4.10 | ..... | 17.80 | " | 26.55 | 1.001 | ..... | ..... | | | | ..... | ..... |
| 94 | ..... | ..... | 39.45* | 93.80 | 90.10 | 3.70 | ..... | 14.60 | " | 23.70 | 1.010 | ..... | ..... | | | | ..... | ..... |
| 95 | ..... | ..... | 37.89* | 93.85 | 90.45 | 3.40 | ..... | 12.50 | " | 21.65 | 1.012 | ..... | ..... | | | | ..... | ..... |
| 96 | ..... | ..... | 36.72* | 93.90 | 90.80 | 3.10 | ..... | 10.70 | " | 19.90 | 1.012 | ..... | ..... | | | | ..... | ..... |
| 97 | ..... | ..... | 34.50* | 94.00 | 91.20 | 2.80 | ..... | 8.00 | " | 17.30 | 1.011 | ..... | ..... | | | | ..... | ..... |
| 98 | ..... | ..... | 31.49* | 94.05 | 91.75 | 2.30 | ..... | 5.50 | " | 14.85 | 1.016 | ..... | ..... | | | | ..... | ..... |
| 99 | ..... | ..... | 30.04* | 94.10 | 92.00 | 2.10 | ..... | 4.30 | " | 13.70 | 1.011 | ..... | ..... | | | | ..... | ..... |
| 100 | ..... | ..... | 28.58* | 94.15 | 92.25 | 1.90 | ..... | 1.95 | " | 11.40 | 1.055 | ..... | ..... | | | | ..... | ..... |
| 101 | ..... | ..... | 25.30* | 94.20 | 92.70 | 1.50 | ..... | 0.60 | " | 9.74 | 1.007 | ..... | ..... | | | | ..... | ..... |
| 102 | ..... | ..... | 24.30* | 94.30 | 92.90 | 1.40 | ..... | 0.15 | " | 9.04 | 1.008 | ..... | ..... | | | | ..... | ..... |
| 103 | ..... | ..... | 26.10* | 94.30 | 92.70 | 1.60 | ..... | 0.40 | " | 10.00 | 1.029 | ..... | ..... | | | | ..... | ..... |
| 104 | ..... | ..... | 28.58* | 94.25 | 93.35 | 1.00 | ..... | 2.00 | " | 11.55 | 1.049 | ..... | ..... | | | | ..... | ..... |
| 105 | ..... | ..... | 30.72* | 94.20 | 92.90 | 1.30 | ..... | 4.55 | " | 14.05 | 1.022 | ..... | ..... | | | | ..... | ..... |
| 106 | ..... | ..... | 32.70* | 94.10 | 91.50 | 2.60 | ..... | 6.15 | " | 15.55 | 1.034 | ..... | ..... | | | | ..... | ..... |
| 107 | ..... | ..... | 34.50* | 94.00 | 91.10 | 2.90 | ..... | 9.00 | " | 18.30 | 1.006 | ..... | ..... | | | | ..... | ..... |
| 108 | ..... | ..... | 35.68* | 94.00 | 91.00 | 3.00 | ..... | 9.95 | " | 19.25 | 1.014 | ..... | ..... | | | | ..... | ..... |
| 109 | ..... | ..... | 38.05* | 93.90 | 90.80 | 3.20 | ..... | 13.00 | " | 22.20 | 1.031 | ..... | ..... | | | | ..... | ..... |
| 110 | ..... | ..... | 39.95* | 93.80 | 90.60 | 3.80 | ..... | 15.05 | " | 24.15 | 1.014 | ..... | ..... | | | | ..... | ..... |
| 111 | ..... | ..... | 41.80* | 93.75 | 89.65 | 4.20 | ..... | 17.70 | " | 26.75 | 1.008 | ..... | ..... | | | | ..... | ..... |
| 112 | ..... | ..... | 43.10* | 93.70 | 89.20 | 4.50 | ..... | 18.50 | " | 27.80 | 1.022 | ..... | ..... | | | | ..... | ..... |
| 113 | ..... | ..... | 44.12* | 93.70 | 88.90 | 4.80 | ..... | 20.60 | " | 29.60 | 1.011 | ..... | ..... | | | | ..... | ..... |
| 114 | ..... | ..... | 44.80* | 93.65 | 88.65 | 5.00 | ..... | 22.30 | " | 31.26 | 0.999 | ..... | ..... | | | | ..... | ..... |
| 115 | ..... | ..... | 46.03* | 93.60 | 88.20 | 5.40 | ..... | 24.10 | All air. | 33.00 | 0.999 | ..... | ..... | | | | ..... | ..... |
| 116 | ..... | ..... | 46.74* | 93.60 | 88.30 | 5.30 | ..... | 23.00 | " | 31.90 | 1.010 | ..... | ..... | | | | ..... | ..... |
| 117 | ..... | ..... | 45.74* | 93.60 | 88.40 | 5.20 | ..... | 23.30 | " | 32.20 | 1.003 | ..... | ..... | | | | ..... | ..... |
| 118 | ..... | ..... | 44.12* | 93.65 | 88.85 | 4.80 | ..... | 21.30 | " | 30.25 | 1.000 | ..... | ..... | | | | ..... | ..... |
| 119 | ..... | ..... | 44.12* | 93.65 | 88.85 | 4.80 | ..... | 21.30 | " | 30.25 | 1.000 | ..... | ..... | | | | ..... | ..... |
| 120 | ..... | ..... | 42.60* | 93.70 | 89.30 | 4.40 | ..... | 18.20 | " | 27.20 | 1.018 | ..... | ..... | | | | ..... | ..... |
| 121 | ..... | ..... | 42.60* | 93.70 | 89.30 | 4.40 | ..... | 18.10 | " | 27.10 | 1.020 | ..... | ..... | | | | ..... | ..... |
| 122 | ..... | ..... | 40.42* | 93.75 | 89.85 | 3.90 | ..... | 16.20 | " | 25.25 | 1.003 | ..... | ..... | | | | ..... | ..... |
| 123 | ..... | ..... | 40.42* | 93.75 | 89.85 | 3.90 | ..... | 15.90 | " | 24.95 | 1.002 | ..... | ..... | | | | ..... | ..... |
| 124 | ..... | ..... | 38.41* | 93.80 | 90.30 | 3.50 | ..... | 13.20 | " | 22.30 | 1.014 | ..... | ..... | | | | ..... | ..... |
| 125 | ..... | ..... | 38.41* | 93.80 | 90.30 | 3.50 | ..... | 13.20 | " | 22.30 | 1.014 | ..... | ..... | | | | ..... | ..... |
| 126 | ..... | ..... | 36.22* | 93.90 | 90.80 | 3.10 | ..... | 10.40 | " | 19.60 | 1.020 | ..... | ..... | | | | ..... | ..... |
| 127 | ..... | ..... | 36.22* | 93.90 | 90.80 | 3.10 | ..... | 10.40 | " | 19.60 | 1.020 | ..... | ..... | | | | ..... | ..... |
| 128 | ..... | ..... | 34.05* | 94.05 | 91.35 | 2.65 | ..... | 7.20 | " | 16.80 | 1.013 | ..... | ..... | | | | ..... | ..... |
| 129 | ..... | ..... | 33.33* | 94.00 | 91.40 | 2.60 | ..... | 7.20 | " | 16.80 | 1.025 | ..... | ..... | | | | ..... | ..... |
| 130 | ..... | ..... | 32.09* | 94.05 | 91.65 | 2.40 | ..... | 6.10 | " | 15.45 | 1.018 | ..... | ..... | | | | ..... | ..... |
| 131 | ..... | ..... | 32.09* | 94.05 | 91.65 | 2.40 | ..... | 6.10 | " | 15.45 | 1.018 | ..... | ..... | | | | ..... | ..... |
| 132 | ..... | ..... | 30.72* | 94.10 | 91.90 | 2.20 | ..... | 4.20 | " | 13.60 | 1.030 | ..... | ..... | | | | ..... | ..... |
| 133 | ..... | ..... | 30.72* | 94.10 | 91.90 | 2.20 | ..... | 4.20 | " | 13.60 | 1.030 | ..... | ..... | | | | ..... | ..... |
| 134 | ..... | ..... | 28.58* | 94.15 | 92.25 | 1.90 | ..... | 2.18 | " | 11.63 | 1.045 | ..... | ..... | | | | ..... | ..... |
| 135 | ..... | ..... | 28.58* | 94.15 | 92.25 | 1.90 | ..... | 2.18 | " | 11.63 | 1.045 | ..... | ..... | | | | ..... | ..... |
| 136 | ..... | ..... | 26.10* | 94.20 | 92.60 | 1.60 | ..... | 0.48 | T. & M air. B. water. | 9.974 | 1.030 | ..... | ..... | | | | ..... | ..... |
| 137 | ..... | ..... | 26.10* | 94.20 | 92.60 | 1.60 | ..... | 0.48 | " | 9.974 | 1.030 | ..... | ..... | | | | ..... | ..... |
| 138 | ..... | ..... | 25.20* | 94.25 | 92.75 | 1.50 | ..... | 0.30 | " | 9.844 | 1.001 | ..... | ..... | | | | ..... | ..... |
| 139 | ..... | ..... | 25.20* | 94.25 | 92.75 | 1.50 | ..... | 0.30 | " | 9.844 | 1.001 | ..... | ..... | | | | ..... | ..... |
| 140 | ..... | ..... | 26.10* | 94.20 | 92.60 | 1.60 | ..... | 0.52 | " | 10.114 | 1.023 | ..... | ..... | | | | ..... | ..... |
| 141 | ..... | ..... | 26.10* | 94.20 | 92.60 | 1.60 | ..... | 0.52 | " | 10.014 | 1.028 | ..... | ..... | | | | ..... | ..... |
| 142 | ..... | ..... | 28.58* | 94.15 | 92.25 | 1.90 | ..... | 2.05 | All air. | 11.50 | 1.051 | ..... | ..... | | | | ..... | ..... |
| 143 | ..... | ..... | 28.58* | 94.15 | 92.25 | 1.90 | ..... | 2.05 | " | 11.50 | 1.051 | ..... | ..... | | | | ..... | ..... |
| 144 | ..... | ..... | 30.00* | 94.10 | 92.00 | 2.10 | ..... | 3.90 | " | 13.30 | 1.026 | ..... | ..... | | | | ..... | ..... |
| 145 | ..... | ..... | 30.00* | 94.10 | 92.00 | 2.10 | ..... | 3.90 | " | 13.30 | 1.026 | ..... | ..... | | | | ..... | ..... |
| 146 | ..... | ..... | 32.70* | 94.10 | 91.60 | 2.50 | ..... | 6.20 | " | 15.60 | 1.012 | ..... | ..... | | | | ..... | ..... |
| 147 | ..... | ..... | 32.70* | 94.10 | 91.60 | 2.50 | ..... | 6.20 | " | 15.70 | 1.025 | ..... | ..... | | | | ..... | ..... |
| 148 | ..... | ..... | 34.50* | 94.00 | 91.20 | 2.80 | ..... | 7.95 | " | 17.26 | 1.036 | ..... | ..... | | | | ..... | ..... |
| 149 | ..... | ..... | 34.50* | 94.00 | 91.20 | 2.80 | ..... | 7.95 | " | 17.25 | 1.036 | ..... | ..... | | | | ..... | ..... |
| 150 | ..... | ..... | 36.22* | 93.95 | 90.85 | 3.10 | ..... | 10.00 | " | 19.25 | 1.029 | ..... | ..... | | | | ..... | ..... |
| 151 | ..... | ..... | 36.22* | 93.95 | 90.85 | 3.10 | ..... | 10.00 | " | 19.25 | 1.029 | ..... | ..... | | | | ..... | ..... |
| 152 | ..... | ..... | 39.45* | 93.90 | 90.20 | 3.70 | ..... | 12.95 | " | 22.15 | 1.045 | ..... | ..... | | | | ..... | ..... |
| 153 | ..... | ..... | 39.45* | 93.90 | 90.20 | 3.70 | ..... | 12.95 | " | 22.15 | 1.045 | ..... | ..... | | | | ..... | ..... |
| 154 | ..... | ..... | 39.95* | 93.80 | 90.00 | 3.80 | ..... | 14.90 | " | 24.00 | 1.017 | ..... | ..... | | | | ..... | ..... |
| 155 | ..... | ..... | 39.95* | 93.80 | 90.00 | 3.80 | ..... | 14.90 | " | 24.00 | 1.017 | ..... | ..... | | | | ..... | ..... |
| 157 | ..... | ..... | 42.20* | 93.75 | 89.45 | 4.30 | ..... | 17.50 | " | 26.55 | 1.021 | ..... | ..... | | | | ..... | ..... |
| 158 | ..... | ..... | 42.20* | 93.75 | 89.45 | 4.30 | ..... | 17.50 | " | 26.55 | 1.021 | ..... | ..... | | | | ..... | ..... |
| 159 | ..... | ..... | 43.00* | 93.70 | 89.20 | 4.50 | ..... | 19.10 | " | 28.10 | 1.011 | ..... | ..... | | | | ..... | ..... |
| 160 | ..... | ..... | 43.00* | 93.70 | 89.20 | 4.50 | ..... | 19.10 | " | 28.10 | 1.011 | ..... | ..... | | | | ..... | ..... |
| 161 | ..... | ..... | 44.48* | 93.65 | 88.75 | 4.90 | ..... | 21.30 | " | 30.25 | 1.008 | ..... | ..... | | | | ..... | ..... |
| 162 | ..... | ..... | 44.48* | 93.65 | 88.75 | 4.90 | ..... | 21.30 | " | 30.30 | 1.010 | ..... | ..... | | | | ..... | ..... |
| 163 | ..... | ..... | 46.03* | 93.60 | 88.20 | 5.40 | ..... | 24.00 | " | 32.90 | 1.001 | ..... | ..... | | | | ..... | ..... |
| 164 | ..... | ..... | 46.03* | 93.60 | 88.20 | 5.40 | ..... | 24.00 | " | 32.90 | 1.001 | ..... | ..... | | | | ..... | ..... |

end of the central portion above referred to, and a man-hole cut into the trunk from the outside between these bulk-heads, gave access to the outside of the venturi during the experiments.

In these experiments the head-gauges were hook-gauges, measuring water-levels inside of stout boxes, that were connected with the 9-foot trunk by ¾-inch pipes. These ¾-inch pipes were connected with shor brass ajutages let into the shell of the trunk, and smoothly filed off on the inside. The trunk itself was tapped to receive these ajutages, in the case of each gauge, at the top of one of the smaller or spigot rings, about 1 foot up-stream from its entrance into one of the larger or bell rings.

The quantity passing the meter was measured over the weir of the testing flume. This is a permanent weir, having a wrought-iron sharp edge, which can be used without end-contractions on a length of about 20 feet, or with end-contractions on shorter lengths.

It was used with end-contractions on a 6-foot length in experiments (1–7), and without end-contractions in the remaining experiments. About 10 feet up-stream from the weir is a horizontal perforated brass tube set some 9 inches above the floor (which in turn is 5.9 feet below the crest of the weir), this tube being connected with a galvanized iron bucket set in a recess in the wall and fitted with a hook-gauge for measuring depths upon the weir. I have always used a light leveling-rod, ¾-inch square, graduated on all four sides, by Darling, Brown & Sharpe, of Providence, R. I., brass-tipped at the ends and set directly on the hook, to compare weir heights with the setting of the hook-gauge. Two racks deaden the water before it reaches the line of brass tube that leads to the hook-gauge bucket.

As some of the weir heights exceed the limit of the Francis experiments, I used in the computations of these quantities the co-efficients given in Hamilton Smith's "Hydraulics."

These are the result of a careful sifting and digest of all attainable original publications of the records of reliable experiments on the discharge over weirs, inclusive of those made by James B. Francis, and recorded in "Lowell Hydraulic Experiments." Their results, differ from the results of the Francis formula in the present instance as follows, giving a few characteristic differences.

| Depth on the Weir. | Difference to Reduce F. Quantity to S. Quantity. | |
|---|---|---|
| | 6-foot Weir. | 20-foot Weir. |
| | Per cent. | Per cent. |
| 0.2 | + 1.7 | + 2.0 |
| 0.8 | ± .0 | — 0.7 |
| 1.0 | + .1 | — 0.45 |
| 2.0 | + 1.0 | + 1.0 |
| 2.5 | + 1.4 | + 1.5 |

The measurement of leakages was conducted with the same care that obtained in the June experiments already spoken of. They could in all cases be measured by the variations in the water-level of a defined area, the proper allowance being then made for an increased or a diminished head upon the orifices causing leakage for the duration of each experiment; in one case, that of the two bulk-heads either side of the venturi, the leakage, an insignificant quantity, was pumped out and measured in pails. The total leakage never exceeded 0.72 cubic feet per second, and ranged from that down to 0.47, or from about 0.3 to 4 per cent. respectively, of the quantity passing the meter.

In these experiments, also, the discharge of the meter was a submerged one. The water at the extreme down-stream end of the 9-foot trunk always stood higher than the top of the trunk.

Before setting the Venturi meter into the 9-foot trunk, but after the head-gauges subsequently used in the Venturi meter experiments had been established, a series of experiments were made to test the loss of head in the original 9-foot trunk. The results of this series are given in the table which follows.

The area was taken = 57.823 square feet as above stated.

$D = 8.58$, though the "average shape" was an ellipse and not a circle.

$l = 152.88$, being the distance between centers of piezometric brass ajutages above spoken of.

Each experiment is based upon the average result of not less than forty consecutive half-minute readings.

In this and the following series of experiments, taking their supply of water from the upper level canal at Holyoke, an extra gate tender was stationed on duty to keep the upper level steady. By constantly wasting out of the upper level, at a point some distance from the head-

gates, then keeping the water steady at this point by regulating the amount wasted, and with the aid of a canal a mile long and averaging 125 feet wide, as was the case, this was reasonably well accomplished. During the course of a single experiment the canal seldom varied as much as 0.10 in level. Then, in the well-known formula $v = n\sqrt{rs}$, being equal to $\frac{h}{l}$ (see Hamilton Smith's "Hydraulics," p. 271,)

$v$, $h$ and $n$ have the following corresponding values.

| $v$ | $h$ | $n$ |
|---|---|---|
| Feet per second. | Feet. | Co-efficient. |
| 0.5 | 0.0012 | 121.9 |
| 1.0 | .0049 | 120.6 |
| 1.5 | .0128 | 111.9 |
| 2.0 | .0238 | 109.4 |
| 2.5 | .0375 | 109.0 |
| 3.0 | .0548 | 108.2 |
| 3.5 | .0763 | 107.0 |
| 4.0 | .1012 | 106.2 |
| 4.5 | .1295 | 105.6 |

The results of the thirteen experiments made, plotted so regularly, and the points were so close together, that there is hardly a choice in reliability between points taken from plotted curves at regular intervals, as given in the table, and the direct results of experiment. I judge from the disagreement of the results above given, with those found at other places, but on longer tubes, either that piezometers do not correctly indicate the $h$ of the formula (see Hamilton Smith's "Hydraulics"), or else that a uniform and non-accelerative régime of the flow of water through the trunk had not become established in the comparatively short length at command for purposes of measurement. This latter circumstance would, in most cases, prevent any attempt to compute the flow of water through mill trunks, and in many cases of city water pipes, by the use o the formula $v = n\sqrt{rs}$, or of any other formula for the discharge of pipes; whose general co-efficients can only be established for the case of a perfectly uniform, permanent flow; three modifying conditions, namely,

those of perfection, uniformity and permanency, which are very difficult to obtain in practice.

Passing now to the description of the table about to be given:

Columns 1 to 8 will hardly need explanation other than that already above written. All the data tabulated are the averages of consecutive half-minute readings, except in case of the head on the weir, where the usual one-minute interval between readings was adhered to.

Columns 9 and 10 will be clear, when it is remembered what was above said with regard to depression of the piezometric water column in the branch of the suction-pipe which leads directly out of the air-chamber, and concerning the vacuum-gauge formed by the other, descending leg of the same suction-pipe; remembering, moreover, that the plane of division between the action of these two forms of the venturi gauge lies on grade 90.909, as above stated.

Column 11 brings us to a peculiarity in this method of metering water, first revealed by this series of experiments, viz., that the indications of the venturi depression, or vacuum gauge, are different, according as the venturi has been pierced upon a different diameter.

As these experiments were made at the expense of and in the interests of the Holyoke Water-Power Company, and for the sole purpose either of discovering or of perfecting a new and practical method of gauging water. I have not pursued the study of this apparent idiosyncrasy of the meter any further than as stated in the Table. As I translate the results, they mean that the venturi must, in all cases, be pierced for connection with the air-chamber vertically at its crown, and may be pierced radially at as many additional points as we please, without affecting the reading of the standard crown orifice. I have no experimental results to guide me in a choice between several venturi orifices and air-chambers (one at the crown always included), or only a crown orifice; or between air-chambers separate and distinct for each orifice, as in the October experiments, and an air-chamber common to several orifices and encircling the venturi, as in the June experiments. Still, as my feeling on the subject, being that of a person who has worked with the two forms of meter, may be interesting, I will state that, at present, I should favor the general form of air-chamber and of orifices used in the June experiments.

Column 12 will need no explanation. In this set of experiments, with the glass suction-pipe next the venturi and the separate air-chambers, these pet-cocks were of subordinate value. To open a pet-cock, so long as the depression-gauge (not vacuum-gauge) is acting, does not disturb the piezometric water column.

Columns 13 to 16 demand no further explanation than was given for the similar columns in the table relating to the June experiments.

As regards the range or oscillations of the several gauges, in the space of one experiment, during this series, it was:

| | | | | | |
|---|---|---|---|---|---|
| In head-gauge No. 1, seldom so much as | | | | .....0.05 | feet. |
| " " " 2, | " | " | " | .....0.05 | " |
| In the depression-gauge, | " | " | " | .....0.11 | " |
| In the vacuum-gauge | " | " | " | ..0.30 to 0.50 | " |

as a result of ½-minute readings, and with no attempt to record absolute maxima and minima.

The quantity of water discharged must have been very nearly uniform in its flow per second.

I pass now to a brief discussion of the results as found in Tables No. 1 and 2.

As a measure of comparison of the uniformity of the results found with the two Venturi meters experimented on in June and in October, 1887, I suggest the range of co-efficients known to exist in case of the weir, or of a simple orifice. The weir has hitherto been regarded as a standard method of gauging water; yet every one who has practiced with it knows how carefully all its dimensions must be proportioned, and the water led to it, in order that it may give truthful or accurate results. The range of the co-efficients entering into any proposed formula for the discharge over a weir, can be and has been limited by limiting the general and proportional dimensions of the apparatus and water depths to which the formula was to be applicable. And the most positive results are undoubtedly found by taking such a limited formula, constructing the weir or other apparatus in accordance with the dictates of the experiments on which it was founded, and then, practically, by repeating the experiments, repeating the attainment of the original results. And wherever this can be done with a weir, the method of the experiments made at Lowell by James B. Francis, and the formula based upon them will, no doubt, long remain the standard method and formula for weir measurements. But without such close limitation and imitation, or taking depths upon the weir ranging only from 0.3 to 2.0 feet, and taking weirs both with and without end-contractions, the co-efficient varies from .660 to .580 (see Plate VII, Hamilton Smith's "Hydraulics"), or 12 per cent.

Taking only weirs without end-contractions, the range is from .660 to .614, or about 7 per cent. In case of the 1 foot Venturi meter, and velocities through the venturi, ranging from 5 to 50 feet per second, this range of co-efficient was 6.5 per cent.; in case of the 9-foot Venturi meter, and velocities through the venturi, ranging from 5 to 36 feet per

# TABLE No. 2.

## HERSCHEL ON VENTURI WATER METER,

### Venturi Experiments, October, 1887.

Observations were taken every half minute; 16 minutes' "Duration," therefore equal 33 observations, and similarly in all the experiments.

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No. of experiment. | Duration of experiment in minutes. | Quantity of water passing venturi. | Velocity through the venturi. | Head gauges. Upper. | Head gauges. Lower. | $H$ = Total head. | Co-eff't = $\frac{\text{Velocity}}{\sqrt{2gH}}$ | Depression gauge. | Vacuum gauge—Height of water column. | Valves. | Pet cocks. | $H_v$ = Head on venturi. | Co-eff't = $\frac{\text{Velocity}}{\sqrt{2gH_v}}$ | Theoretical $H_v = \frac{H}{H'_v}$ | Theoretical co-efficient. |
| | | Cub. ft. per sec. | Feet per second. | Feet. | Feet. | Feet. | | Feet. | Feet. | Nos. open. | | Feet. | | Feet. | |
| 1 | 16. | 12.40 | 1.75 | 99.8830 | 99.8783 | 0.0047 | 3.188 | 99.8441 | ..... | 1 | ..... | 0.0389 | 1.108 | 0.0397 | 1.097 |
| 2 | 10. | 12.36 | 1.75 | .8765 | .8714 | .0051 | 3.051 | .8489 | ..... | 5 | ..... | .0276 | 1.311 | | |
| 3 | 10. | 12.40 | 1.75 | .8930 | .8875 | .0055 | 2.947 | .8538 | ..... | 3 | ..... | .0392 | 1.104 | | |
| 4 | 20. | 35.10 | 4.96 | .8820 | .8374 | .0446 | 2.929 | .4981 | ..... | 1 | ..... | .3839 | 0.998 | .3897 | 0.991 |
| 5 | 20. | 54.02 | 7.64 | .8383 | .7361 | .1022 | 2.978 | 98.9209 | ..... | 1 | ..... | .9174 | .994 | .9311 | .987 |
| 6 | 10. | 53.98 | 7.63 | .8433 | .7415 | .1018 | 2.982 | .9268 | ..... | All | ..... | .9165 | .994 | .9302 | .986 |
| 7 | 10. | 53.91 | 7.62 | .7966 | .6948 | .1018 | 2.978 | .8803 | ..... | 5 | ..... | 9163 | .993 | | |
| 8 | 20. | 71.11 | 10.05 | .8403 | .6652 | .1751 | 2.995 | .2117 | ..... | 5 | ..... | 1.6286 | .982 | | |
| 9 | 9.5 | 70.99 | 10.03 | .8333 | .6584 | .1749 | 2.992 | .2072 | ..... | 1 | ..... | 1.6261 | .981 | 1.6496 | .974 |
| 10 | 9. | 70.80 | 10.01 | .8250 | .6525 | .1725 | 3.005 | .2018 | ..... | All | ..... | 1.6232 | .979 | 1.6467 | .972 |
| 11 | 20. | 88.76 | 12.55 | .7731 | .4998 | .2733 | 2.993 | 97.2222 | ..... | " | ..... | 2.5509 | .980 | 2.5878 | .973 |
| 12 | 20. | 103.11 | 14.58 | .7090 | .3490 | .3600 | 3.029 | 96.2773 | ..... | " | ..... | 3.4317 | .981 | 3.4815 | .974 |
| 13 | 20. | 119.96 | 16.96 | .6358 | .1400 | .4958 | 3.003 | 94.9873 | ..... | " | ..... | 4.6485 | .981 | 4.7158 | .974 |
| 14 | 19. | 139.15 | 19.67 | .5278 | 98.8574 | .6704 | 2.995 | 93.1935 | ..... | " | ..... | 6.3343 | .974 | 6.4246 | .968 |
| 15 | 20. | 162.46 | 22.96 | .4179 | .5125 | .9054 | 3.009 | ..... | 0.2495 | " | ..... | 8.7584 | .968 | 8.8820 | .961 |
| 16 | 8.5 | 162.60 | 22 98 | .4309 | .5246 | .9063 | 3.010 | ..... | .0042 | 5 | ..... | 8.5261 | .981 | | |
| 17 | 9. | 162 60 | 22.98 | .4124 | .5071 | .9053 | 3.012 | ..... | .2516 | 1 | ..... | 8.7550 | .969 | 8.8786 | .962 |
| 18 | 9. | 162.60 | 22.98 | .4094 | .5037 | .9057 | 3.011 | ..... | .0011 | 2 and 6 | ..... | 8.5015 | .983 | | |
| 19 | 4.5 | 162.60 | 22.98 | .4129 | .5129 | .9000 | 3.021 | ..... | .2400 | 1, 2 and 6 | ..... | 8.7439 | .969 | 8.8675 | .962 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | 4.5 | 162.60 | 22.98 | .4033 | .5015 | .9018 | 3.018 | ..... | .2300 | 1 | No. 1 air<br>No. 2 water. | 8.7243 | .970 | 8.8479 | .963 |
| 21 | 20. | 183.90 | 26.00 | .2618 | .1097 | 1.1521 | 3.020 | ..... | 2.7156 | 1 | 1, 2 and 6, 3 and 7 air<br>Nos. 4 and 8, 5 water | 11.0684 | .974 | 11.2266 | .967 |
| 22 | 9. | 183 62 | 25.96 | .2710 | .1173 | 1.1537 | 3.013 | ..... | 2.6979 | 1, 2 and 6, 3 and 7 | ..... ..... | 11.0599 | .973 | 11 2171 | .966 |
| 23 | 9.5 | 183.62 | 25.96 | .2391 | .0996 | 1.1395 | 3.032 | ..... | 2.3189 | 2 and 6, 3 and 7 | ..... | 10.6490 | .992 | | |
| 24 | 4. | 183.90 | 26.00 | .2618 | .1204 | 1.1414 | 3.034 | ..... | 2.6689 | 1, 2 and 6, 3 and 7 | ..... | 11.0217 | .976 | 11.1799 | .969 |
| 25 | 4.5 | 183.48 | 25.94 | .2447 | .0976 | 1.1471 | 3 019 | ..... | 2.7160 | 1 | ..... | 11.0517 | .973 | 11.2089 | .966 |
| 26 | 20.5 | 207 24 | 29 30 | .1200 | 97.6337 | 1.4863 | 2.996 | ..... | 6.1874 | All | All air | 14.3984 | .963 | 14.5988 | .956 |
| 27 | 19. | 207.39 | 29.32 | .1391 | .6466 | 1.4925 | 2.992 | ..... | 6.1982 | 1 | ..... | 14.4283 | .962 | 14.6298 | .956 |
| 28 | 26. | 207.09 | 29.27 | .1330 | .6475 | 1.4855 | 2.995 | ..... | 5.9514 | 5 | No. 5 air | 14.1754 | .969 | | |
| 29 | 10.5 | 206.79 | 29.23 | .1174 | .6297 | 1.4877 | 2.988 | ..... | 6.0986 | 4 | No. 4 air | 14.3070 | .964 | | |
| 30 | 4. | 206.95 | 29.25 | .1089 | .6290 | 1.4799 | 2.998 | ..... | 6.1389 | 4 and 8 | No. 8 air | 14.3388 | .963 | | |
| 31 | 10. | 206.64 | 29.21 | .1052 | .6192 | 1.4860 | 2.988 | ..... | 6.0505 | 3 and 7 | Nos. 3 and 7 air | 14.2467 | .965 | | |
| 32 | 5.5 | 206.64 | 29.21 | .1015 | .6265 | 1.4750 | 2.999 | ..... | 5.7827 | 2 and 6 | Nos. 2 and 6 air | 13.9752 | .974 | | |
| 33 | 2. | 206.95 | 29.25 | .1062 | .6288 | 1.4774 | 3.001 | ..... | 5.8080 | 2 | ..... | 14.0052 | .975 | | |
| 34 | 3.5 | 207.39 | 29.32 | .1209 | .6386 | 1.4823 | 3.002 | ..... | 5.8762 | 6 | ..... | 14.0881 | .974 | | |
| 35 | 8.5 | 206.95 | 29.25 | .1301 | .6421 | 1.4880 | 2.990 | ..... | 6.1668 | 1 | No. 1 air | 14.3879 | .962 | 14.5883 | .955 |
| 36 | 19.5 | 217.51 | 30.75 | .0299 | .3933 | 1.6366 | 2.997 | ..... | 7.6815 | All | All air | 15.8024 | .964 | 16.0234 | .958 |
| 37 | 14.5 | 217.51 | 30.75 | .0095 | .3837 | 1.6258 | 3.007 | ..... | 7.7983 | 1 | ..... | 15.8988 | .961 | 16.1198 | .955 |
| 38 | 13.5 | 217.65 | 30.77 | .0322 | .4094 | 1.6228 | 3.011 | ..... | 7.7157 | 5 | No. 5 air | 15.8389 | .964 | | |
| 39 | 5. | 217.65 | 30.77 | .0222 | .4019 | 1.6203 | 3.014 | ..... | 7.8409 | 1 and 5 | ..... | 15.9541 | .960 | 16.1751 | .954 |
| 40 | 3.5 | 217.51 | 30.75 | .0231 | .3971 | 1.6260 | 3.006 | ..... | 7.5550 | 3 and 7 | Nos. 3 and 7 air | 15.6691 | .968 | | |
| 41 | 5. | 218.45 | 30.88 | .0546 | .4315 | 1.6231 | 3.022 | ..... | 7.6427 | 4 and 8 | Nos. 4 and 8 air | 15.7883 | .969 | | |
| 42 | 20. | 232.83 | 32.91 | 98.9249 | .0596 | 1.8653 | 3.005 | ..... | 10.2205 | All | ..... | 18.2364 | .961 | 18.4901 | .954 |
| 43 | 10. | 233.31 | 32 98 | .9515 | .0829 | 1.8686 | 3.008 | ..... | 10.2314 | 1 | ..... ..... | 18.2739 | .962 | 18.5289 | .955 |
| 44 | 9. | 233.00 | 32.94 | .9462 | .0774 | 1.8688 | 3.004 | ..... | 9.9032 | 5 | ..... | 17.9404 | .970 | | |
| 45 | 16.5 | 233 16 | 32.96 | .9670 | .1002 | 1.8668 | 3.008 | ..... | 10.0259 | 3 and 7 | Nos. 3 and 7 air | 18.0839 | .966 | | |
| 46 | 12. | 245.84 | 34.75 | .9134 | 96.8216 | 2.0918 | 2.996 | ..... | 12.1940 | All | ..... | 20.1984 | .964 | 20.4805 | .957 |
| 47 | 14. | 243.87 | 34.47 | .7734 | .7241 | 2 0493 | 3.003 | ..... | 12.2045 | 1 | ..... | 20.0689 | .959 | 20.3471 | .953 |
| 48 | 9.5 | 243.54 | 34.43 | .7593 | .7077 | 2.0516 | 2.997 | ..... | 12.0130 | 5 | ..... | 19.8633 | .963 | | |
| 49 | 4. | 244.52 | 34.56 | .8234 | .7536 | 2.0698 | 2.996 | ..... | 12.1822 | All | ..... | 20.0966 | .961 | 20.3761 | .955 |

second, it was 3.3 per cent. But better than this is the fact that the two meters, though differing so much in their size and structure, showed a total combined range of co-efficient no greater than the smaller one alone, or only 6.5 per cent. Taking the co-efficients based on $H_x'$, being those corrected for velocity of approach, and using that simplest of all hydraulic formula, $v = \sqrt{2g\,H_v'}$.

Though the areas of discharge were as 81 to 1, and the interior fractional surfaces were widely different, the resultant co-efficients are at extreme points only 6.5 per cent. apart; and the deviation of any single experiment from the resultant mean is 3.0 per cent. in case of the 1-foot tube, excluding the unreliable interpolated results, and only ½ per cent. in case of the 9-foot trunk for its whole range of velocities.* If we compare this to the case of a discharge through various orifices, the result is still more gratifying. To the wearied sojourner among such tables of discharge—ranging in their co-efficients from the familiar 0.6 or ⅔, up to the mystical co-efficients in the eighty's and ninety's, said to have been found by some one "on large sluice-gates in France" (and occasionally met with in the current practice of the hydraulic engineer)—a consistency in co-efficients as above found for the hydraulic apparatus herein described, is indeed refreshing. We appear to have here, at last, an apparatus for gauging liquids which may range in its dimensions, its materials of construction and in manner of use, so as to cover all ordinary practice, and yet have only 6.5 per cent. of range of co-efficient, at the same time requiring only the simplest of observations and of formulas to work with. Or by limiting the use of the meter to velocities greater than 9 feet per second through the venturi, being about 1 foot per second through the pipe thereto appurtenant, all the ranges of variations above given become materially less.

I said above that we "appear" to have such an apparatus, or, to completely express the underlying thought, we appear to have it in the light of the only two sets of experiments yet made, so far as I know, with a Venturi meter set in line of a pipe. But further experiment will be needed to confirm or upset such a conclusion, and one object of the present paper is to invite such further experiments.†

* See Plate XXXIX, showing plotted results.

† While this paper is being written I am in receipt of the October number of the *Journal of the Association of Engineering Societies*, which gives the results of experiments upon similar forms of discharge, but discharging from a tank, and through an orifice of only about 0.03 feet in diameter, and in which the same co-efficient is likewise found nearly equal to 1.

The reason, I will suggest, why the co-efficients belonging to this form of gauging apparatus are so nearly uniform, is largely on account of the close similarity between the conditions assumed by theory and those found in actual practice, regarding now the state of the liquid as it passes through the venturi. Here, if anywhere, water may be supposed to flow as though composed of the traditional "filaments" of the school-books; while the bubblings of a boiling, seething caldron are but little more violent and irregular than the motions of the alleged "threads" of water, as we find that water in ordinary practice, and as it flows in canals or even in the ordinary line of pipes, or in tubes.

Still the co-efficient is not the same for all velocities; it is less for higher velocities than it is for lower ones in the October experiments while the reverse holds true in the June experiments; the meter does not appear to be applicable for velocities below 5 feet per second* through the Venturi, or of about ½ foot per second through the pipe in which it is placed; and the co-efficent is not equal to one, except in one instance.

The difference between the equation giving the locus of the co-efficients as applicable to the 9-foot trunk and to the 1-foot tube, may be due to difference in asperity of their interior surfaces; some of it may possibly be due to the shortening of the 9-foot cones, caused by the trunk measuring only 8.7 feet high instead of 9 feet as supposed.

Part of the deficiency from the value 1. may be due to defective guidance of the water as it approaches the venturi. It would have been better, no doubt, to have rounded off the angle with which the up-stream end of the smaller cone meets the up-stream pipe or trunk; better still, to have made that portion of the meter up-stream from the venturi of a form which would be generated by the revolution about the central axis of an ogee curve. In the case of the discharge from an open canal or from a tank, this portion of the meter could be suppressed entirely, and in its stead be placed only a mouth-piece, having the form of the contracted vein to feed the venturi; with a head-guage reading directly the water-level in the tank or canal.

It is also an interesting question whether the vacuum-gauge is indica-

---

* For the case of $v = 0$, $Hv$ is also $= 0$, and the co-efficient becomes $= \frac{0}{0}$ which may be any assignable quantity.

This justifies the curiously diverging form of the curves of co-efficients for the two Venturi meters, as shown on the diagram.

tive of the mean velocity, or of the velocity of the exterior filaments of the body of water passing through the venturi, or of both, and what is the precise meaning of the readings of this and other piezometers tapped into pipes of flowing water. In our present very imperfect knowledge of the action and precise meaning of the indications of such piezometric columns, especially when applied to tubes, but little can be positively affirmed about them.*

Loss of head is still the only difficulty to contend against in the practical application of the meter for mill purposes. For purposes of metering a city, or domestic water supply, or water used for purposes other than power in mills, this loss is insignificant. In the other case named, and for a 9-foot trunk, it would be about 1 foot, when the mean velocity through the trunk was 2.7 feet, and ½ foot for a velocity of about 1.9 feet. If the circumstances are such that this loss of head is not permissible, or cannot be paid for by the delivery of enough more water to yield to the consumer an equivalent amount of power, then this meter cannot be used in a form that would make it continuously the sole outlet or inlet of the water to be metered. It could be applied in those cases either at the inlet or outlet, in the penstock or in the tail-race, but would have to be provided with some form of byepass to be kept open at all those times when the operation of metering the water was not actually going on. This could be readily done in the case of an open feeder or of most any tail-race, and as the operation of metering need require so little time, barely five or ten minutes, there could hardly be any objection made by the consumer to this plan of measuring water. It probably need not be pointed out that the whole apparatus could literally be submerged, or covered with water, and yet be conveniently used and act as it ought to, so long as it afforded the only outlet from one body of water to another, and that its advantages in freedom from any moving parts, and from liability to be stopped up or put out of order by floating substance or by ice, are very great.

Writing so soon, only a few weeks after the close of the second set of experiments, I do not very likely allude to all the capabilities of the meter, and have hardly broached the interesting subject of the theory of

* For a discussion of the subject, brought down to the present day, see Hamilton Smith's "Hydraulics," London and New York, 1886. Mr. Hiram F. Mills has investigated the case of piezometers connecting with an open trough or conduit. See Vol. VI, 1879. Proceedings of the American Academy of Arts and Sciences.

PLATE XXXIII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVII No 371.
HERSCHEL ON
VENTURI METER.

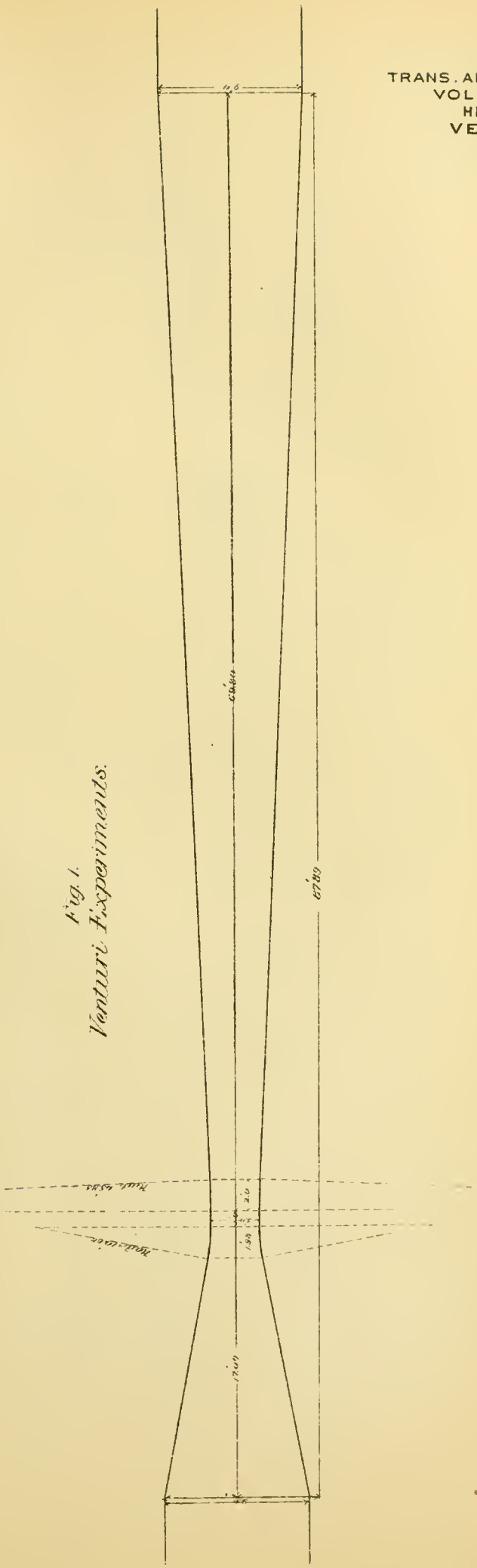

Fig. 2

Venturi Water Meter.

Used June 1887.

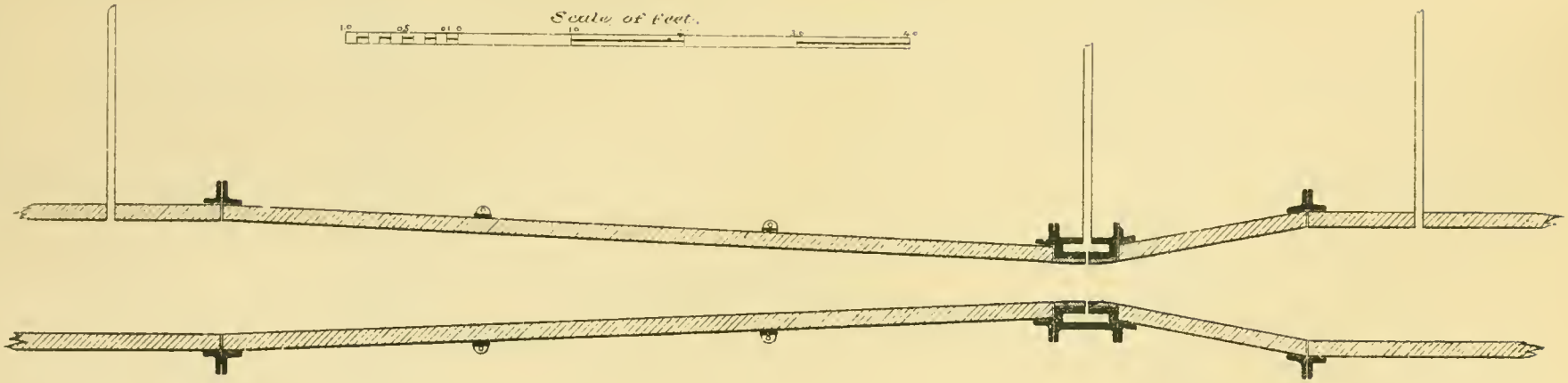


PLATE XXXIV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XVII No. 371.
HERSCHEL ON
VENTURI METER.

PLATE XXXV
TRANS. AM. SOC. CIV ENG'RS.
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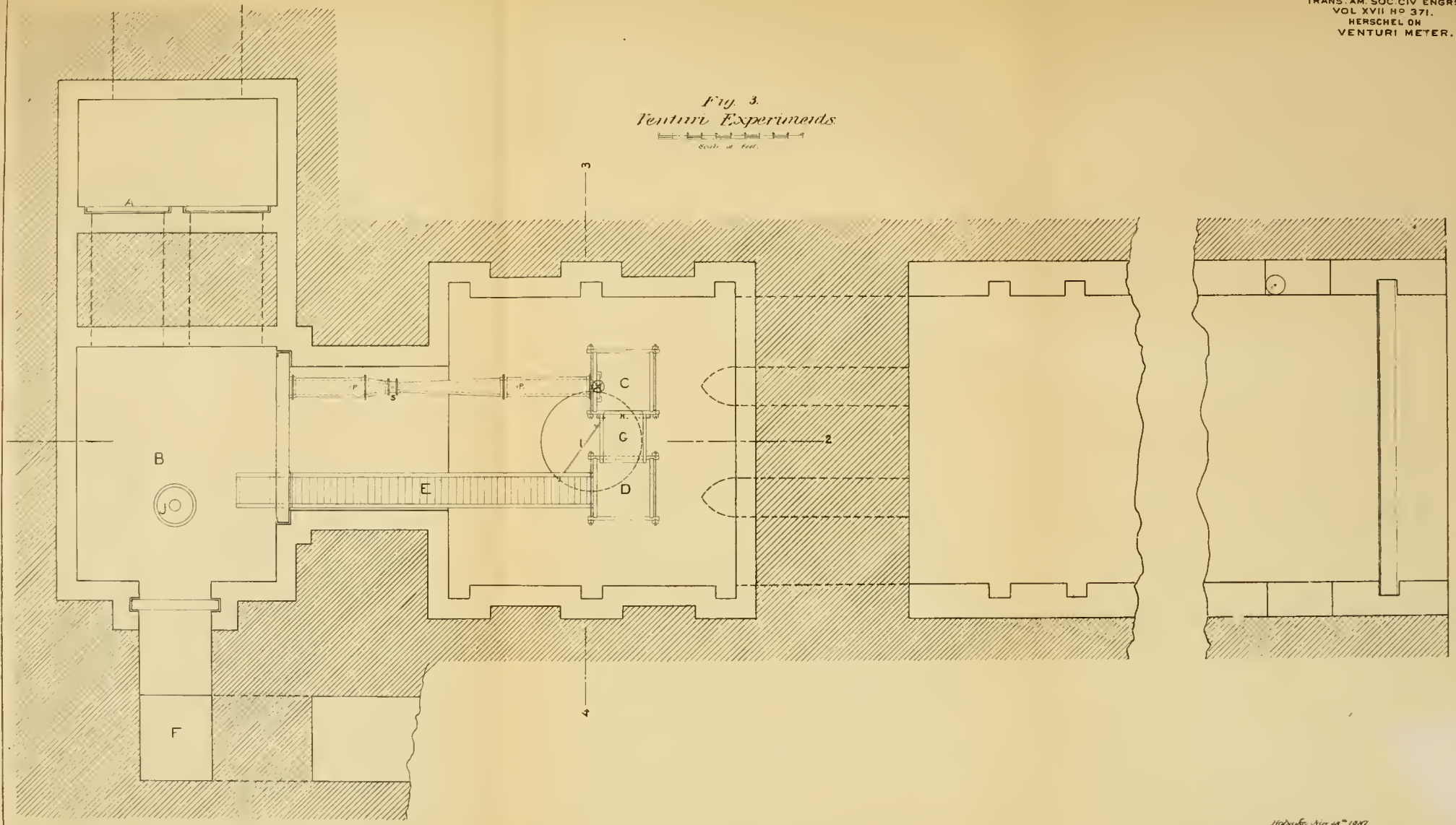
HERSCHEL ON
VENTURI METER.

*Fig. 3.*
*Venturi Experiments.*

Scale of Feet.

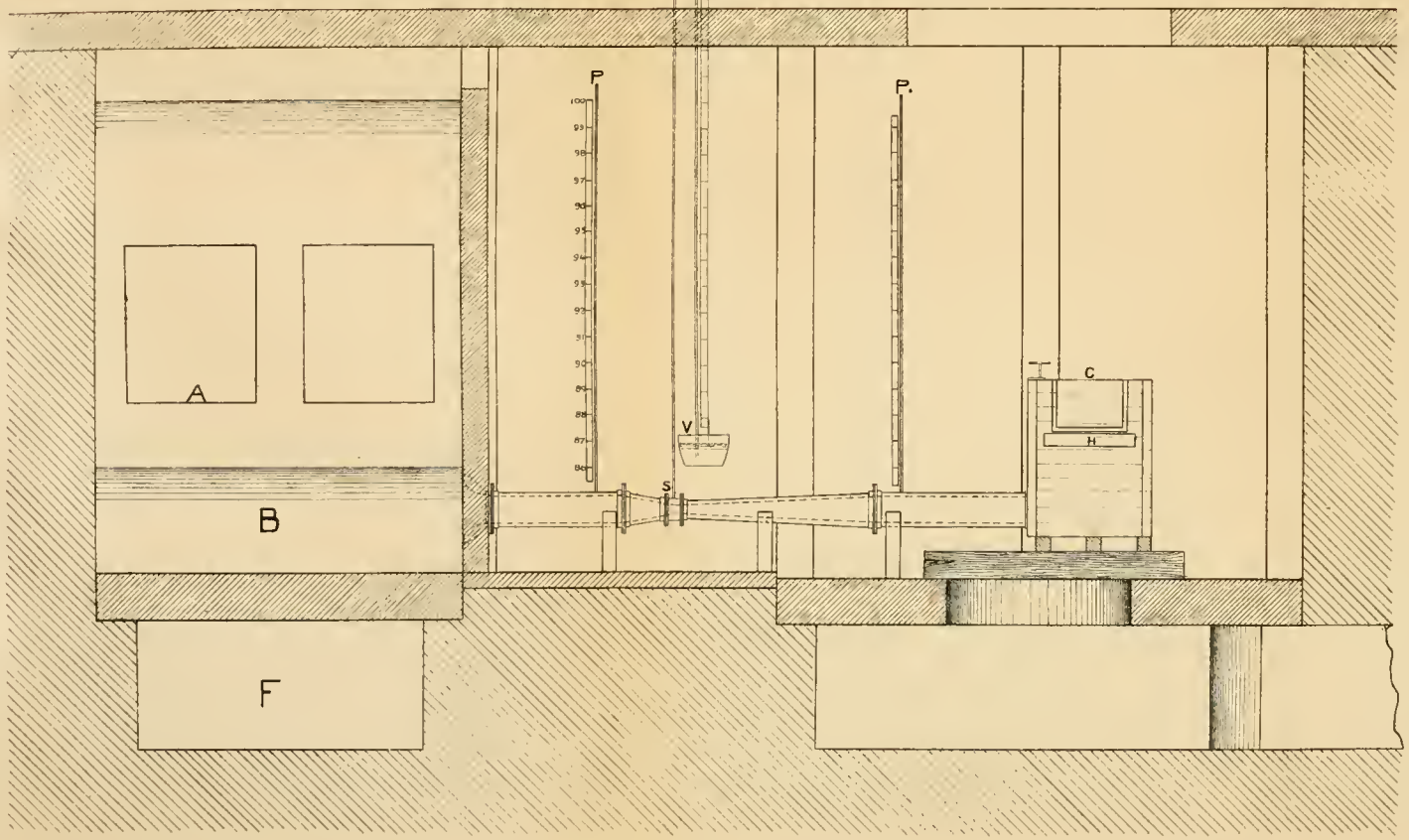



Holyoke Nov 24th 1887

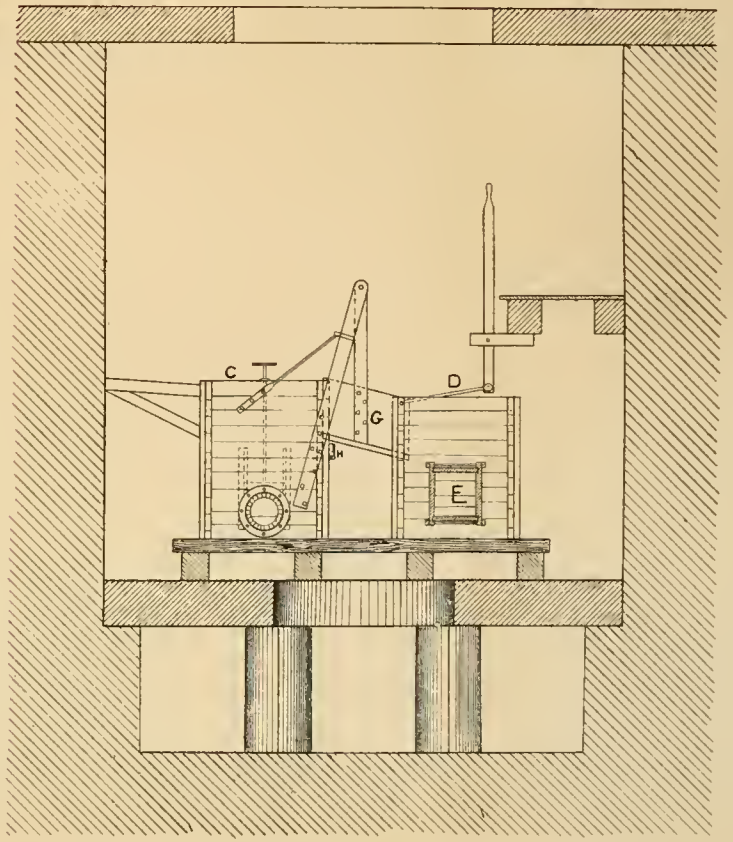
Fig. 4.
Venturi Experiments.

Scale of feet.

Section 1-2.

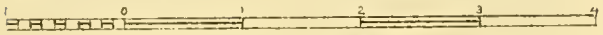



Section 3-4.

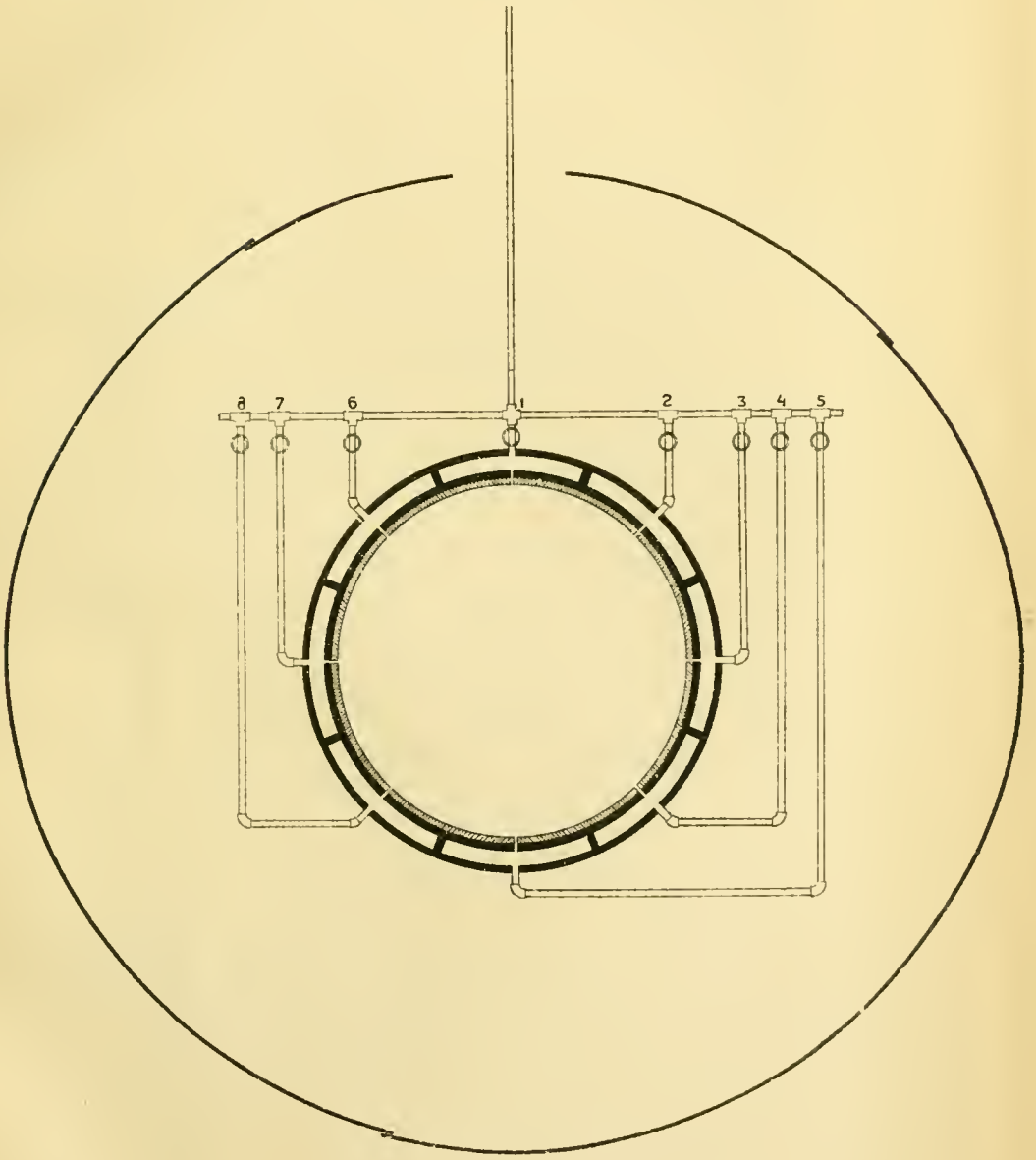



Holyoke Dec 7th 1887

*Fig. 5.*
*Venturi Experiments.*




*Scale of feet.*



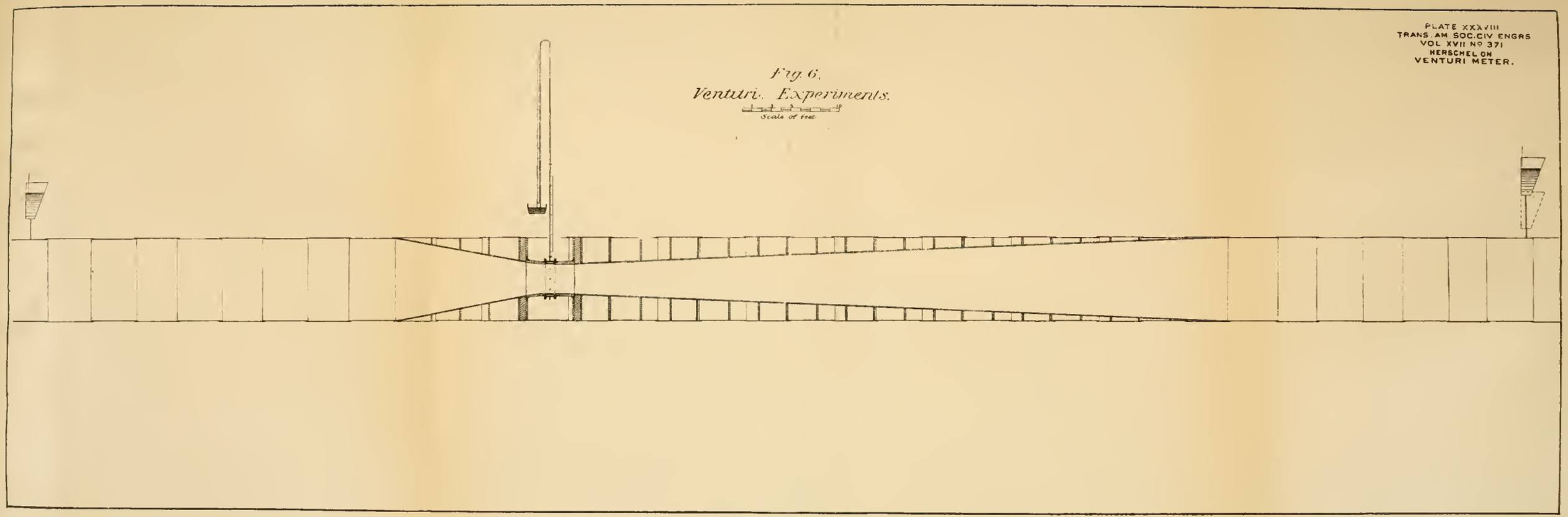


Fig. 6.
Venturi. Experiments.
Scale of feet

the instrument. It seems to me that it may in many instances replace the use of a weir, being easier applied and equally or more accurate, and it can be used where a weir is entirely inapplicable.

To sum up the matter, I will only submit to my associates in the profession that there is the promise of a great and varied usefulness in the Venturi meter. And while I have applied for a patent on it, in the United States, mainly as a protection in the use of it by the Holyoke Water-Power Company and by myself, I shall be pleased to give a limited license to use, in case a patent is granted, to any one who will make and communicate to me the results of further accurate experiments made with it, in the general planning or arranging of which, I hope to be allowed to make some suggestions.

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Mr. Clemens Herschel (In reply to questions asked at the time of reading the paper).—The questions asked, following the reading of the paper on the Venturi water meter, have been very instructive to the author of the paper. They have shown, for one thing, that the immediate interest of the Members is more eagerly turned in search of an account of the practical working apparatus and of directions for its use, than it is towards an account of the principles upon which the apparatus works, together with a precise and detailed record of experiments substantiating the claim of such working. Both forms of information have, however, their value, each in its own time and place, and each supplements the other. In its completed form, the Venturi meter is an instrument to gauge the quantity of water flowing in a pipe, by measurement of an abrupt, artificially produced depression in the hydraulic gradient. To explain more particularly, suppose a pipe full of water, the water in a state of rest. Piezometers placed on a such a pipe, will have the water stand in them at points situated all on one level.

In Plate XL, suppose *PP* to be such a pipe, at one time "submerged," or under a head, to the extent 1 *P*, and again, only to the extent *a P*. Next, suppose the water to take any velocity through the pipe (no meter being yet supposed inserted) sufficient to cause the water in the piezometers to stand on the line 22 and *bb*, respectively, according as the amount of submergence was originally 1 *P* or *a P*. This line 22, or *bb*, is what I have called the "hydraulic gradient." Next, suppose the meter inserted in the pipe; upon which, the water level at the up-stream piezometer will remain at 2, but at the piezometer which is set on the venturi, the water level or hydraulic gradient will drop to 3, then rise again, at the end of the meter, up to within a small distance below its former position (this distance representing the "loss of head" due the whole apparatus), then will run parallel to its former position, as shown at 3

on the down-stream piezometers. The experiments have shown that the velocity of the water, or the discharge through the narrowest section of the meter through the "venturi," is that due the head on the venturi (as represented by the difference in level of two points in the "hydraulic gradient," one taken just above the meter and the other at the venturi), with a co-efficient, which is remarkably constant, whether applied to a rough meter and for a 9-foot pipe, or to a smooth one for a 1-foot pipe, and for all velocities through the pipe ranging from ½ to 6 feet per second. If this co-efficient is taken without further measurement at 98 per cent., we may be certain from experiments so far made that we shall rarely be over 2 per cent. out of the way. Going back a little, let us take now the other case of submergence, originally represented by the hydraulic gradients *aa* and *bb*. It is plain that the water level in the piezometer which is set on the venturi cannot fall below the surface of the stream spouting through the venturi.

But so much of the "depression" in the hydraulic gradient at this point due the velocity of the water, which lies below the surface of the stream just named, will be indicated or exhibited by the sucking action or aspiration or "vacuum" spoken of in the paper; and in measures of a column of water lifted, will exactly equal that portion of the "depression," as shown in Plate XL (or as it may be computed), which lies below the surface of the stream spouting through the venturi. In Plate XL it is equal to the distance *v c*, indicated by the reference marks.

This is the case in which some form of vacuum-gauge is necessary at the venturi, when separate gauges are used at the up-stream piezometer and at the venturi, as was done during the experiments related in the paper. No such complication is necessary, however, in practice. As the measure sought is the difference of pressure immediately above the meter and at the venturi, a single pressure-gauge suffices. The logical possibilities, depending on the degree of submergence of, and velocity through the pipe, are three, and are exhibited by the table:

| | Above the Meter. | At the Venturi. |
|---|---|---|
| 1 | Pressure. | Pressure. |
| 2 | Pressure. | Vacuum. |
| 3 | Vacuum. | Vacuum. |

But in any event there will be a "head on the venturi," or, what is the same thing, Column 1 minus Column 2 of the tabular quantities will always be positive, and will indicate pressure, and may be measured by a single-pressure gauge.

The most accurate form of pressure-gauge suitable for this purpose will be a simple "manometer tube" or "*U*-gauge."

Two small pipes would be connected with the meter (one entering the water pipe just above the meter proper, the other attached to the venturi), the other ends of these pipes to be connected with the two arms of the *U*-gauge; upon which, the quantity passing, in cubic feet per second, or in any other measure, could be read off directly on the scale attached. By making the connecting pipes of ample size, the gauge could be placed 1 000 feet, or more if need be, from the meter. The only difference in the gauges made to fit the several sizes of meters would be in the scale. I believe this will answer many of the questions asked respecting the working of the meter, and may also make clearer the principles involved.

The question was asked, Can the accompanying gauge be made self-recording? And after the exercise of a little inventive skill, I am pleased to say that I have no doubt it can.

I have refrained from looking to the construction of the cheap, but not entirely reliable or accurate forms of pressure-gauge, to indicate differences of pressure, which could be made by using some form of apparatus containing a piston or floats and stuffing boxes, and have adhered to the ordinary manometer mercury gauge as the apparatus for measuring the difference of pressure in question. For the extreme velocity of 6 feet per second through the pipe (54 feet per second through the venturi), a length of 3.5 feet for the mercury tubes in the gauge is more than enough.

As regards the effect on the pressure-gauges of the surface and of the interior velocities of the stream passing through the venturi, I have no further speculation to offer. The difference between surface and interior velocities must be about as small in the "wire-drawn" stream of water spouting through the venturi, with nine times the average velocity that the water has in the main pipe, as in any known form of hydraulic apparatus.

Answering question which relate to the temperature of the water during the experiments recorded in the paper, I will state that this varied from 67 to 71 degrees Fahr. during the June experiments, with the temperature of the air in the wheel-pit, and of the water in the tub of the pressure-gauge, varying from 66 to 71 degrees.

During the October experiments the temperature of the water was 57–57.5 degrees.

It is undoubtedly true that all hydraulic formulas are affected by the temperature of the water, when that temperature passes beyond those ordinarily found in running water; but in ordinary practice, and without reference to water artificially heated, and as it is at times found in steam engineering practice, no account has ever been or need be taken of temperature that the author knows of.

The formula used in the computations, it will be observed, supposes a discharge through an orifice, under pressures crudely represented by the hydraulic gradient.

A more scientific way would have been to make use of the hydraulic principle first enunciated by Dubuat, but disputed by Navier and others, that the pressure against any point in the walls of any vessel or pipe is always equal to that of the contained fluid, supposed to be in a state of rest, less the height due the velocity past that point.

Or, passing to algebraic symbols, if

$P$ be the pressure in terms of the height of a column of water at the point $P$, Plate XL, and

$P_1$ " " at the point $V$ of Plate XL ; if

$v$ " velocity " " $P$, and

$v_1$ " " " " $V$; also

$P_s$ " pressure if the water be supposed to be still; then

$$P = P_s - \frac{v^2}{2g}$$

$$P_1 = P_s - \frac{v_1^2}{2g}$$

and $v_1 = 9\ v$, from the construction of the meter.

Subtracting the equations, we have:

$$P - P_1 = \frac{v_1^2}{2g} - \frac{v^2}{2g} = \frac{80}{81}\frac{v_1^2}{2g}$$

But $P - P_1$ is what we have called $H_v$, the "head on the venturi." Or

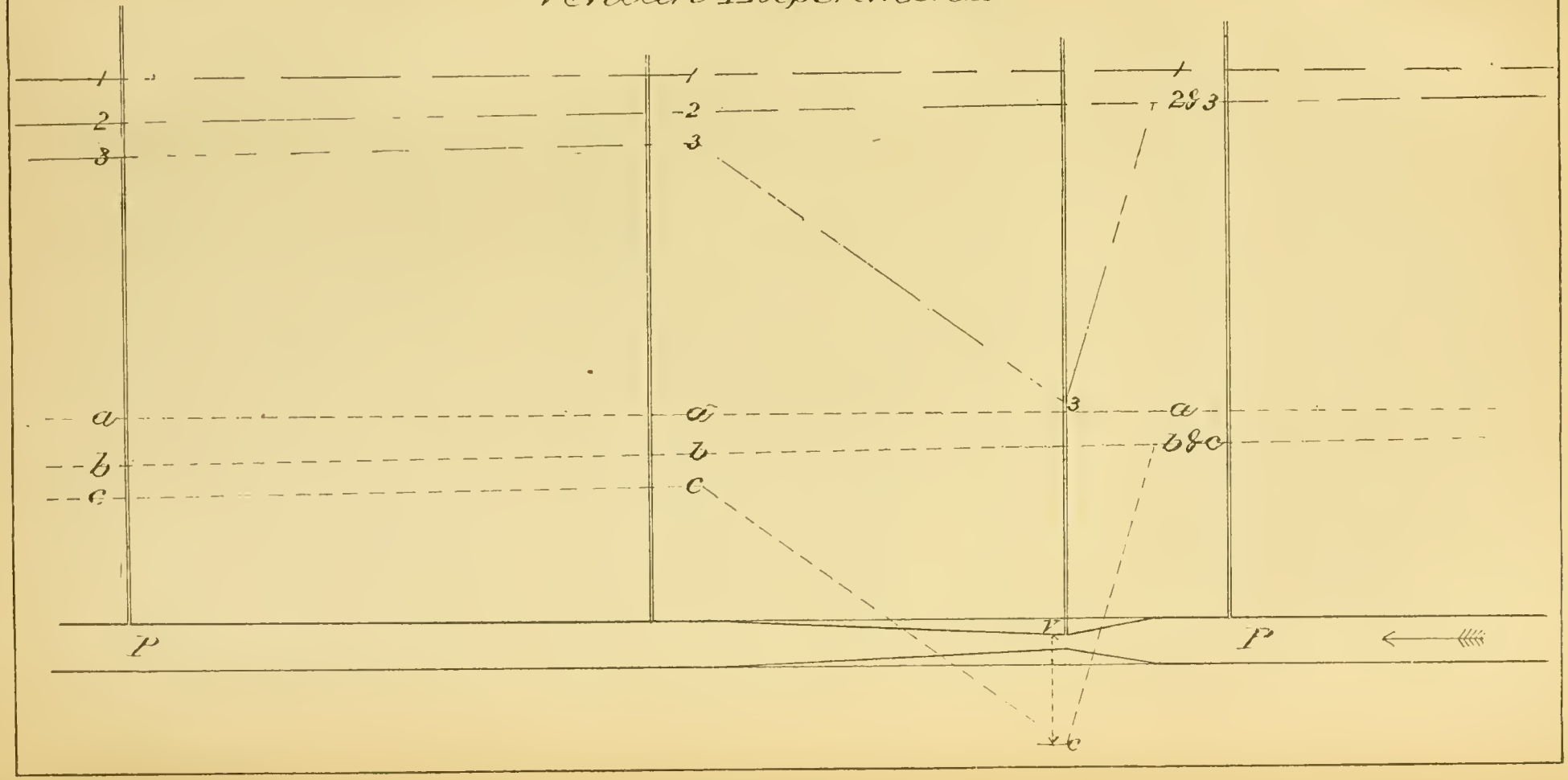
$v_1$ = the velocity through the venturi,

$$= \sqrt{\frac{81}{80}}\sqrt{2\,g\,H_v} = 1.0062\sqrt{2\,g\,H_v};$$

where the first supposition has supposed $v_1 = \sqrt{2\ g\ H_v}$, abstracting in both instances from the co-efficients for actual use, which experiment alone can supply.

Fig. 7.

Venturi Experiments.



AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVII.—December, 1887.)

# INSPECTION AND MAINTENANCE OF RAILWAY STRUCTURES.

A DISCUSSION AT THE ANNUAL CONVENTION, JULY 2D–8TH, 1887.

By JOHN A. WILSON, WILLARD S. POPE, GEORGE H. PEGRAM, G. BOUSCAREN, W. S. LINCOLN, JOSEPH M. WILSON, JAMES G. DAGRON, J. A. L. WADDELL, EDWARD S. PHILBRICK, A. BRYSON, ROBERT A. SHAILER, W. A. HAVEN, JOHN M. GOODWIN, ALBERT LUCIUS, E. P. DAWLEY, C. A. MARSHALL, D. J. WHITTEMORE, THEODORE COOPER, H. STANLEY GOODWIN, A. M. WELLINGTON, PERCIVAL ROBERTS, JR., C. C. SCHNEIDER, C. FRANK ALLEN, JOHN BOGART, FREDERIC GRAFF, ROBERT I. SLOAN, J. FOSTER FLAGG, J. J. R. CROES, WILLIAM KENT, ROBERT MOORE AND WILLIAM J. MCALPINE.

At the request of a number of members of the Society, a Discussion upon the Inspection and Maintenance of Railway Structures was announced for the Convention of 1887. A circular was prepared and issued, embodying a number of suggestions that had been made on the subject, and a request was made for discussion upon these and such other points as might occur to the members taking part in the discussion. The suggestions were as follows:

*First.*—What measures, legal or other, can be taken to insure a proper inspection of railway bridges?

*Second.*—What is proper bridge inspection?

*Third.*—Should there not be a standard specified rolling load much heavier than as now generally used, and a specified engine wheel base for rolling loads?

*Fourth.*—Is it not expedient to adopt a standard bridge floor?

*Fifth.*—Should not bridges of small span be made strong enough for a buckle-plate floor and a continuous coat of ballast on the bridge, and if so, up to what span should this apply?

*Sixth.*—Should not a safety guard (Latimer), be used at all openings over a certain width?

*Seventh.*—Should there not be required either overhead crossings, or, in their place, interlocking apparatus with derailing switches?

*Eighth.*—Is legislation as to any of these points, or as to any others you may suggest, expedient, and if so, what sort of legislation?

*Ninth.*—In this connection the experience of the Master Car Builders' Association has been referred to, which it is stated, has proved that the action of large committees reporting to the Association, and the adoption of standards after ample discussion, have been found very valuable.

## DISCUSSION BY LETTERS.

John A. Wilson, M. Am. Soc. C. E.—I have just received the circular relative to the maintenance and inspection of railway structures. The time is so short that I am not able to give this important subject the thought which it deserves and requires, and can only touch briefly on some of the points of inquiry.

Question 2.—Assuming that reference is intended to bridges already in use, and not to new bridges to be built, I understand by proper bridge inspection, such examinations and oversight of a bridge by competent parties, as will insure the immediate detection of any defect or signs of failure or disarrangement of any of the parts of the structure, so that the condition of the structure may be positively known at all times.

Take a railroad bridge which has been properly designed and constructed. The bridge engineer having it in charge should have in his office full detailed plans of the structure, with strain sheets and calculations, and should know just what load the bridge will carry with safety. He should personally make a critical examination of the structure at least once a year, and oftener if his judgment indicates it to be necessary, and should hold himself in readiness to make a special examination at any time. He should keep a record of his observations, and of the reports made to him by his subordinates. A competent master carpenter or foreman of bridges should keep himself thoroughly posted as

to the condition of the structure, examining it once a month or oftener if necessary, and report in detail to the engineer. The road foreman or watchman should make a daily examination, and any defects that he may discover should bo promptly reported to the master carpenter. When the engineer makes his examinations he should have the master carpenter and watchman or road foreman with him, confer freely with them on practical points, and see that they clearly understand their duties, what to look out for, and how to report. By this system the structure is under constant surveillance, and any defect should be detected immediately.

Where bridges have been erected and used for a considerable time without any oversight, considerable labor is involved in putting into operation a proper system of inspection. The bridges should be measured, examined and calculated, and full plans made, so that the engineer may have in his office exact information on which to base his operations. Without this, there can be no certainty about his work.

It is assumed as a matter of course, that when defects have been discovered the proper remedy will be applied.

In this connection it may not be out of place for me to say, that while we have in this country good bridging concerns who employ competent engineers, and turn out only first-class work, there are undoubtedly others which need the closest professional oversight, from the very commencement, to secure good designs, good material and good workmanship. The business of bridge building is free to all; the majority of people know nothing about it; they are told they shall have a good bridge, and when a bridge is finished and handsomely painted they assume that it is all right, while it may have the most glaring defects, and contain within itself the elements of speedy destruction. Much of the trouble with bridges in this country is due to original defects of design, material or workmanship, which no system of inspection can remedy, but can only point out. It would therefore seem as if inspection, to be complete, should include new bridges about to be constructed as well as those already existing.

Further, it should be noted that the designing engineers of bridges deal mostly with new structures which may be all right when erected, but may be put to uses for which they never were intended, and at best, will wear out in time like anything else. The maintenance of such structures under use is a business in itself, which can be learned only by practical experience. It is a business which the engineer who only builds bridges, and has no care of or responsibility for them afterwards, knows little about, whence the necessity for the "engineer of maintenance for bridges."

It is such an "engineer," in my judgment, that should have charge of the duties described above. He should not only watch the structure and discover its defects from time to time, but be able intelligently to

apply the necessary remedy, and no small part of his duty will be to see that loads are not placed on the structure which it never was intended it should carry.

QUESTION 3.—It would be difficult to establish an absolute standard of rolling load for the whole country, as the requirements and character of traffic vary on different railroads. Each railroad should establish for its own line a standard high enough to cover all contingencies, have the bridges conform thereto, and then make it a rule not to put more work on bridges than they are calculated for. The general tendency all over the country among transportation men, is, to increase the weight and dimensions of their rolling stock. They should establish a reasonable limit to which the engineer can work. The through bridges and tunnels now existing are the only things which control the height and width of passenger cars.

QUESTION 4.—It would certainly be well to decide on the best design of bridge floor, and have it brought into general use throughout the country.

QUESTION 5.—I am not prepared to answer this question. There are some practical difficulties in the way of adopting such a plan generally.

QUESTION 6.—It is very desirable to use a safety guard on bridges wherever practicable.

QUESTION 7.—My views on this subject are best expressed by the following extract from the evidence which I gave in a grade-crossing lawsuit in 1882.

"Grade crossings of railroads are very numerous in this country. When the country was new and traffic light, railroads were projected for local business, and necessarily had to be constructed at minimum expense, and grade crossings of different lines were allowed without much consideration. As the railroad system of the country has become extended, traffic and the speed of trains have increased, and the conditions of the problem have changed. The evil effects of grade crossings on leading lines have thus become more apparent, and while it is often difficult to change the location of a road after it has been in operation for years, and all the surroundings have accommodated themselves to the existing conditions, there is a growing feeling in the country in favor of a separation of grades—it is being done in some cases on existing roads at heavy expense, and there is a very decided aversion on the part of railroad managers to extend or repeat the evil in new construction.

The objections to a grade crossing of one railroad by another may be summed up under three general heads:

*First.*—Danger to life and property.

*Second.*—Delays to traffic.

*Third.*—Expense of operating.

### I.—Danger to Life and Property.

This item cannot be entirely eliminated under any system of management. With the arrangement of signals now in use comparative safety can be secured—and it is undoubtedly a fact that grade crossings on important lines are successfully worked. Still there is left a percentage of danger—due principally to carelessness of employees or neglect to observe the rules; breakage of machinery about engines or cars on approaching a crossing, rendering it impossible to control the train; or error of judgment on the part of enginemen in approaching crossings under too much headway in bad weather with slippery rails, etc.

### II.—Delays to Traffic.

It may be assumed as a general principle that the more efficient the signal system and the nearer the approach to absolute safety in the operation of a grade crossing, the more will be the delay to traffic. The amount of delay at any given place will depend largely on local circumstances and conditions, but it is nevertheless true that on any important line a grade crossing by another road must cause more or less delay, require a great amount of watchfulness, and be a constant source of anxiety to the officers and employees operating the road.

### III.—Expense.

This item may be separated into two parts:

*A.* Those items which can be estimated in money, viz: Cost of crossing-plates and maintenance of the same. Cost of signal tower, etc., and maintenance of the same. Wages of emyloyees to manage signals, etc., with expenses for fuel, lights, etc.

*B.* Those items which, while appreciable, cannot be estimated in money, viz: Possible losses to life, limb, or property by accident. Value of time that might be lost by trains detained at crossing, and cost of stopping and starting trains, extra fuel consumed, etc. Increase in general operating expenses of road. Injury to rolling stock at crossing places.

A separation of grades should be had wherever it is practicable. If a grade crossing of one road by another cannot be avoided then every known safeguard, such as signals and interlocking apparatus with derailing switches should be provided.

Questions 1 and 8.—The general tendency in this country has been not to control the details of railroad management by legislative enactments, but to leave the railroads free to make their own regulations, holding them pecuniarily responsible for the results, it being supposed

that self-interest will properly regulate the matter. For many reasons it is desirable to follow this policy, but if the desired results are not obtained in this way, it may become necessary to establish legislative control. It is an intricate subject which would require careful study and elaboration, and cannot be hastily covered in a paper like this. Any legislative enactment on this subject should be had only after full discussion and consultation with the leading engineers and railroad men of the country. If a few of the leading railroad companies of the country would introduce a thorough and effective system of bridge inspection, recognizing the importance and necessity for separating the professional oversight of bridge work from its mechanical execution by contractors and bridge-building companies, the manifest advantage would soon be recognized, and public opinion would force the general adoption of the system.

The subject of grade crossings of railroads is one which, perhaps, more properly could be and should be regulated by law. Some of the States have laws on this subject, but they are generally crude, indefinite, and subject to local contingencies for their operation. In New York, a crossing of one road by another is examined into and decided on by a commission of three, one of whom must be a civil engineer. This commission decides whether the crossing shall be over, under or at grade. In Massachusetts, the subject is under the control of the railroad commissioners, who must give their consent before a grade crossing can be made. In Pennsylvania the matter is in equity proceeding before the county court, and the act of Assembly says: "And if in the judgment of such court it is reasonably practicable to avoid a grade crossing, they shall by their process prevent a crossing at grade." The practical effect of this act, is, a long trial, a lot of expert evidence both ways, and in the majority of cases a decision by the court that "it is not reasonably practicable to otherwise construct the crossing." It would be well if a definite policy in this matter could be adopted, with more uniformity in the laws of the several States.

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WILLARD S. POPE, M. Am. Soc. C. E.—*Governmental Supervision of Bridges.*—It is probably generally believed that the State should exercise some intelligent and efficient supervision over all bridges within its domain. So far as is fairly possible the lives and property of its citizens should be safe while crossing such structures. An accident which might have been prevented by reasonable precaution becomes a crime.

It might be supposed that an enlightened self-interest would be a sufficient motive to insure good and reliable construction. When a railroad bridge falls, it costs the company a good deal of money, generally much more than the simple renewal of the structure, and the same

is true in event of disaster to a highway or country bridge. And so it would seem that self-interest ought to be an efficient safeguard. And in many cases it is. But there are far too many instances in which the persons in authority do not know, and are not competent to judge, of the strength and reliability of the structure, and many times even if they are aware of its weakness, they are inclined, from motives of false economy, to trust it a little longer, and to run some risk in order to delay a needed expenditure. And so by one or all of these reasons the life and property of the public are jeopardized. The matter is a reasonably clear one for the intervention of a higher power, and no power is so fit to undertake the responsibility as the Government.

But to act efficiently it must act intelligently. There is nothing mysterious or bewilderingly occult about a bridge. To the trained eye its defects, if any, are patent. But here, if anywhere, the trained eye is essential. Bridge wisdom is peculiarly a specialty. No one is born with it. Genius does not leap to it, nor does blundering stumble upon it. It comes only by patient study and long training. To pass judgment upon the merits of a proposed design for a bridge, or to reach an intelligent idea of the strength and trustworthiness of a structure already in use, requires not only common sense and sound business discretion, but also technical skill and experience in the details of that particular kind of construction, knowledge of the principles of computation of the magnitudes and effects of strains, and information as to the strength and capacity of materials. In brief, it is essentially and exclusively business for a trained expert. Therefore it is apparent that any control which the State may assume over bridges will be vain and useless unless it places the matter in the right hands.

To secure this result there should be, in my judgment, a commission organized by the State, to which should be relegated this whole business. All bridges within the limits of the State, whether for highway or railway purposes, should be included within its jurisdiction. A critical official examination should be had annually, or as often as might be necessary, of all existing structures, and reports be made thereon to the authorities in charge thereof, with such recommendations as the respective cases might seem to demand. If any structure should be found unsafe and the suggestions for repair or renewal were not immediately and properly complied with, the commission should have the power to order it closed to travel, and the town or company owning the same should be responsible in penal damages for any accident or disaster that might occur thereafter. The commission should prepare and publish such general specifications as to character and capacity of new structures as might be advisable under the varying circumstances, and no bridge should be allowed to be built except in conformity with the requirements of those specifications. All plans of proposed new bridges should be submitted to the commission, and receive its formal written

approval before work be commenced thereon, and all work in progress under such plans should be subject to its inspection, both as to quality of material and workmanship; and no new bridge should be opened for public use until it had been examined and approved. Plans of all bridges should be filed and preserved in its office, and these together with all its official proceedings should be always, under proper restrictions, open to the public. General designs might be prepared in detail and published, for spans of ordinary length and dimension, which could be used to advantage in many localities. In fact the operations and publications of the commission could not fail to be of great service in disseminating among those interested, useful and trustworthy information on the general subject of bridge building. Highway commissioners and town supervisors of bridges would find such suggestions and advice of great and permanent value. It would prove a school of technical instruction that would certainly be of much practical service.

In the above catalogue of the duties and powers of the proposed commission I would like it distinctly understood that they are not intended to interefere with or supersede in any way the duties and powers of the owners of bridges, whether towns or companies. The railroads and the town authorities will continue to have all the power and responsibility that they ever had. They will build, maintain and care for their bridges as heretofore, will inspect, watch over and repair them as they have always done. No responsibility will be taken from their shoulders, and none of their rights or powers will be invaded. The duty and authority of the commission is simply to see that the bridges as built or repaired are safe. The operations of the commission go hand in hand with, and not counter to, the operations of the bridge owners. They are friends and coadjutors, not rivals or conflicting authorities. They are to work together for a common end, and under proper management, instead of jealousies and misunderstandings, there will come friendly and hearty co-operation. All that the commission will demand and insist upon is safety, and that it is bound to have and will use all the authority in its hands to secure.

The commissioner or one of his assistants should accompany the railroad engineer or the highway supervisor on his periodical tours of inspection, and the needed repairs or renewals should be matter for free and frank consultation. The railroad managers and the county authorities should forward at once to the commissioner copies of any reports on the condition of bridges made to them by any of their subordinates, and, in short, all parties concerned should and could work together harmoniously and effectively for the common good. A faithful trial for a year or two would, I am sure, convince the most skeptical of the utility and economy of such a commission.

The commission should be headed by a man who is a professional bridge engineer of well-known skill, experience, tact and integrity.

Grave and weighty responsibility would devolve upon him, and he should have corresponding authority. To secure the greatest efficiency he should have the entire and unlimited control of the affairs of his department. He should appoint his own assistants and subordinates, fix their salaries and regulate all the expenditures of the office. If he is worthy of being appointed he is worthy of being trusted, and he should be entirely untrammeled and unhampered by political or other outside influence. He should be in every respect a first-class man, the best that could be got, and in his appointment nothing should be considered except his pre-eminent fitness for the special duties of that special position.

A man possessing such qualifications is not a common one, nor when found is he a cheap one. The salary and the term of office should be such as would justify the right man in accepting the position. He should receive at least ten thousand dollars per annum, and the term of office should be five years, subject, however, to dismissal for cause by the Governor.

The entire yearly cost of such a commission for this State (Michigan), thoroughly and actively administered, would, I suppose, be about forty thousand dollars. One-half of this sum should be paid directly from the State Treasury, and the other half by the railroads, being divided among them in proportion to their track mileage. If the commission were what it ought to be I believe the railroads would willingly assume this charge. The managers realize not only the necessity of having safe bridges, but also the desirability of public confidence in their safety; and surely the knowledge that such a supervision was had by disinterested and competent officials would tend largely to inspire such public confidence. The cost would be comparatively small for any one of the railroads, and they would thereby secure an additional and valuable inspection and supervision. From such a commission they would get help and not hindrance, co-operation and not annoyance. The commissioner and his assistants would be a corps of active and capable consulting engineers, the value of whose services would be worth to the railroads much more than the cost.

If this reasoning is applicable to the railways, it applies with much more force to the town and county authorities in their bridge work. These officers are, for the most part, honest and intelligent men, striving to do their duty to the best of their ability. In ordinary road and street work they are doubtless sufficiently competent, but in the technical details of bridges they are unskilled and uninformed, and they themselves are for the most part willing frankly to admit it. Such being the case, it seems reasonable to suppose that in these matters they would gladly avail themselves of the assistance of such a commission. Their real powers would be unimpaired. They would still decide as to the amounts of money to be appropriated, the method and manner of its

expenditure, the award of contracts, etc. The commission would be advisory, but in no hampering or offensive sense. The combination would work to the assistance of the town officers, and the public could be reasonably sure that it was not cheated by contractors, that it got its money's worth, and that it got good and trustworthy bridges.

So far as the cost of the commission is concerned the estimate of forty thousand dollars a year may seem large. But remember that there is a great amount of work to be done. There is in the State of Michigan nearly six thousand miles of railway, and probably ten times that length of wagon roads, and the aggregate amount of bridging must be very great. To properly look after all this will require a considerable force of men, and they must all be men of skill and professional training. And we should consider the magnitude of the interests at stake. A single bridge disaster may destroy in property, and in many instances has done so, tenfold the whole cost of such a commission, to say nothing of the loss of human life. If the object sought can be certainly secured by an annual expenditure of forty thousand dollars, there should not be a moment's hesitation. Such a sum is a bagatelle in the comparison. Possibly my judgment is at fault as to the precise amount. It may be somewhat more or somewhat less than the sum I mention. The precise amount needed can only be told by trial. There must be money enough to secure a first-class commissioner, and to provide him with an ample corps of first-class assistants, and to pay a considerable amount of traveling expenses, etc. The money which such an organization may cost will be well spent, and the staff will fairly earn every dollar that is paid them.

And now arises the question as to the selection of a commissioner. I think every one will admit that the office should be entirely and forever divorced from politics. It should be utterly beyond the reach of a party leader or a political striker. What is wanted is a bridge engineer, not a politician. Therefore it is manifestly out of the question that he should be subject to election by popular vote. He must be appointed. And who shall appoint him? Not certainly the Legislature. Any objections which apply to a popular election apply equally to such an appointment. The power might perhaps be placed in the hands of the Governor. But the Governor or any other individual or body that is elective might very possibly be swayed by motives more or less tinged by his or their political surroundings and affiliations. And furthermore, however conscientious and well-informed the Governor may be, it is not likely that he would be sufficiently acquainted with the subject or with professional engineers to be able to select the right man. Therefore the Governor is hardly the proper authority to make the choice.

The matter thus narrows itself down towards one and only one direction. The commissioner should be selected and nominated by those who will directly use his services, viz.: by those who have bridges to

build and care for. They best know what is wanted, and their acquaintance with engineers personally and by repute is such that they can most readily put their hands on the right man. They would be intimately brought into contact with him in the various operations of their daily business, and would carry a joint responsibility with him in all official transactions, and consequently would naturally feel the warmest interest in the character and capacity of the man with whom they had thus to deal.

Now, of those who have bridges to build and care for, there are two classes: first, the town and county officers in charge of bridge work, and second, the railroads. In the attempt to choose a commissioner, the first class would be heavily handicapped by reason of their great number, and by the fact that they are scattered all over the State in every township, and are for the most part unknown to each other, and so would probably find it impracticable to act in concert. And thus we come step by step to see that the selection of the commissioner should be by the railroads. And it is eminently fit that it should rest here. The railroads are vitally interested in the matter, and all their natural motives would be simply to get the best man. They operate a great number of bridges all over the State, on the safety of which incessantly depend immense values of property and great numbers of lives. They would pay directly one-half the cost of the commission. Their managers are few in number, acquainted with each other, and can easily act in concert. They are widely known to the whole community as men of character, capacity, intelligence and high standing. Their business connections are such that they are necessarily largely acquainted with engineers, and their means of information are ample as to the ability and standing of any individual. It might be reasonably expected that the man whom they should unitedly, or a majority of them, select would be fit for the place.

Let the Governor appoint the man whom the railroads nominate, put into his hands the duties and powers hereinabove indicated, and I believe a long stride will be made in the direction of public safety and welfare.

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George H. Pegram, M. Am. Soc. C. E.—1. When we consider the number of bridges in this country, the experimental state of bridge building at the time most of them were built, and the rapid increase in loads and speed of trains, the number of accidents seems very small, and those which might have been avoided by inspection in the light of our present knowledge so small that there seems no justification in hampering the railway companies with legal enactments which might introduce greater evils through the divided responsibility which might ensue, particularly when the specification of bridge engineering and

inspection constantly tends to greater security and the railway companies evince a disposition to improve and renew their bridges as fast as can reasonably be expected.

2. Proper bridge inspection can only be made by a person thoroughly acquainted with the principles involved in the design as well as the mechanical manipulations through which the work has passed, and it is much better that such inspection be made at long intervals of time than that the work be subjected to the frequent torture that is now so common by unskillful hands. A complete record of every defect is of the greatest importance in order that chronic weaknesses may be avoided in future designs.

3. The rolling load on bridges is constantly increasing, and a standard would necessarily have to be heavier than that now in use to endure for any length of time. The real difference in the costs of bridges for light and heavy loads within the limits usually specified can be ascertained by railroads by asking for bids on both, and it is probable that when the small proportionate difference is seen the heavier load will generally be adoped. There seems to be no reason for specifying an engine wheel base when better results can be obtained in much less time with a uniform load in combination with a single concentrated load.

4. It does not seem expedient to adopt a standard bridge floor, because the proper floor to use will depend somewhat upon the construction of other parts of the bridge and there is a wide difference of opinion as to what is best.

5. A sufficiently good floor is made without the use of buckle-plates which contain elements of danger in their liability to corrosion and inaccessibility to inspection and painting.

6. An efficient guard rail is of all things the most important part of a bridge, because most failures of bridges are knock-downs. Rails, when used for this purpose, do not prevent the bunching of the ties, and it would seem that the case is most effectively and economically met by a wooden guard rail notched over the ties, and having an angle-iron nosing throughout its length, as I understand is used on the Chicago, Milwaukee and St. Paul Railway. In all cases, the ties should be sufficiently strong and well supported to carry a train when off the track. The addition of rails running to a point at the end of the bridge, as in the Latimer device, is, of course, so much additional security, and ought generally to be used.

7. It would seem desirable to require interlocking apparatus and every other known precaution at all crossings, but hardly practicable to make any limitations as to grade.

8. Legislation on any of the points covered by the suggestions in the circular would seem inexpedient, and a commission with judicial power in the selection of designs very much to be deplored, because the selec-

tion of men with sufficient skill and integrity, and, above all, free from personal bias and political influence, would be almost impossible, and the growth and development, now very rapid, might be seriously checked.

9. The experience of the Master Car Builders' Association in the selection of standards can hardly be properly cited in this connection. The devices discussed by them are nearly the same in cost and comparable in efficiency, while in bridge work it is frequently a matter of how much the road can afford to pay, and the efficiency is generally a matter of judgments born of different experiences. A poor bridge that stands does as effective work as a good one.

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G. Bouscaren, M. Am. Soc. C. E.—I am prevented at the last moment from attending the Convention as I had intended to do. The time is too short now for lengthy remarks, but I am induced to say something in hope that it may bring about a full discussion from the able members present at the Convention; it seems very desirable that something should be done to rectify the course into which we are now drifting.

To question (3) of the circular, I reply, No, for the obvious reason that the weights of locomotives and trains vary within very wide limits from one road to another. Although it is good and prudent policy for every road to provide for the future in the construction of structures designed to be permanent, it can hardly be expected that a province road with light traffic, or a small feeder to a trunk line should make the same allowance in that direction as the main line of a great system with steep gradings and heavy traffic.

My reply to question (4) is that all bridge floors should be impenetrable to a derailed train; but to specify a uniform floor standard for all roads would be inexpedient for the same reason given against a uniform standard load.

Question (5). I would recommend ballasted floors on all bridges where creosoted ties are used, but would not put ballast on unprepared timber, and would prefer creosoted ties to buckle plates. The elastic medium offered by the wooden ties is, I think, very essential.

Question (6). Guard rails should be used on all bridges, and the Latimer guard or an equivalent substitute at the ends of all through bridges and of all high deck bridges at least.

I reply in the affirmative to question (7) which properly relates, however, to the subject of railroad crossings. For reasons already stated, arising from the great difference in the character and requirements of different roads, I would not consider it expedient and desirable that fixed standards of load, wheelbase, floor and guards be legislated, but I think it is of the greatest importance that proper inspection be enforced by the States or by the general Government.

This inspection should not consist in an "intelligent walk" over the structure or a "good glance" from the rear platform of a Pullman car, but in the verification of the facts as to whether a bridge is, or is not, properly adapted to the purpose for which it is being used: or in other words, whether it is properly designed and proportioned, first to carry safely the maximum load to which it is actually subjected; second, to resist the abnormal effects of accidental causes, such as extraordinary wind storms and the derailment of trains, and whether it is being properly maintained, as to repair and adjustment of parts, and the protection of its material against natural deterioration.

We all know why so many poor structures are to be found on some otherwise first-class roads, but it may be well to speak out the reasons here.

Let us assume the most favorable circumstances: a road is being built under the supervision of an engineer who, perchance, happens to know something about bridges. He prepares his specifications for the heaviest load used in the country, and adds something to it to provide for contingent increase in the future; this does not suit the president of the company, who is a practical man; he does not propose to build a road on the model of the Pennsylvania or the Baltimore and Ohio; he wishes to provide only for the traffic in sight, and is not going to have heavy engines to bend his rails—he directs the engineer to change his load. The road is built, the engineer is dispensed with of course; a carpenter is in charge of the bridges; he can frame and put up a Howe truss, but he is not "at home" with his iron bridges—they rattle some and are getting rusty—he screws them up!

The weights of engines and trains on the road are increased fifty per cent. by the force of circumstances, and the bridges are still standing, but for how long? This is the early history of a great many American railroads. Now comes on other trouble, it first began with steel rails. What is wanted are rails that will not break like glass or crush like lead. Engineers have tried to learn what little is known with regard to steel and to utilize this knowledge in specifying some of the qualities which appeared essential to have in the steel they were buying. Manufacturers have taken exception to this and have so resolutely and unanimously set their heads together against this presumption of the engineer, that it is next to impracticable now to have rails made on specifications here as is universally practiced in Europe; perhaps this may have a bearing on the marked difference which has been apparent lately in the lasting qualities of American and European rails.

A movement in the same direction is now being made by some bridge manufacturers: they have specifications of their own which they propose to substitute for engineers' specifications; some deal with the quality of materials only, others with the workmanship, load, and the proportioning of parts as well, and the proposition sometimes takes the shape of an ultimatum, "You shall have this or nothing."

Grant the same degree of unanimity and cohesion to the bridge builders on that ground, as obtains for steel rail manufacturers and there shall be no more need for the engineer, except, perhaps, as experts called upon to unravel the mysteries of bridge wrecks, before a coroner's court. I do not wish to ruffle the feelings of my friends, the manufacturers, nor to impeach their ability, nor to impute their motives, but I believe it is an unsound principle and an ill-advised business move to dispense with the intermediacy of the engineer between the buyer and the contractor. It may suit the purpose of contracting companies admirably well, but I believe it is not calculated to promote the interests of the public, the progress of the art, nor the high repute of the profession, yet this is the drift of the situation, and I could not suggest a better remedy than rigid legislation on the proper inspection of railroad bridges.

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W. S. Lincoln, M. Am. Soc. C. E.—In reply to your letter of inquiry I desire to submit the following replies to your inquiries:

Question 2.—In addition to the daily inspection made by the bridge watchman, an inspection should be made every thirty days by a careful man skilled in erection and accustomed to the care of tracks, approaches, etc., who should make careful report of the condition of the whole structure as far as it may be apparent to him. An additional inspection should be made by a competent engineer at least every six months, who should carefully examine every part of the structure and make reports to the Chief Engineer.

Question 3.—I am inclined to the opinion that a standard specified rolling load should be adopted and also specified engine load and wheel load.

Many of the roads have adopted what would seem at present to be in excess of what is required, but as the practice seems to me to increase the weight of the rolling stock and loading it must be considered a step in the proper direction and as "taking the safe side." It may be urged by some of the advocates of lighter loads that many of the smaller roads having light engines do not require such heavy bridges. The live load from the general car loading will be the same for every road in the country, and if the heavy engines are economical and desirable for the larger roads, they will ultimately come into general use, and there should be no more permanent iron work built for light loads.

Question 4.—I think it would be expedient to adopt a standard bridge floor, and enclose a plan, Plate XLI, of such floor as I have had in use for many years.

Question 5.—I see no necessity for a buckle-plate floor and am not in favor of it. It is objectionable for various reasons—difficulty of making

repairs—in the way of painting and inspection—increasing liability to corrosion.

QUESTION 6.—The Latimer or some equally effective appliance should be used at all openings over 20 or 30 feet depending somewhat upon location.

QUESTION 7.—Crossings of other roads should be overhead when possible. If this is not practicable, interlocking apparatus with the derailing switches should be used, and this should be enforced by proper legislation, though as to this, each road should certainly take its own precautions as to the safety of its structures and should be held responsible for the results. The inspection should be thorough and made by a competent engineer. A superficial examination, such as is frequently made, should not be permitted, and, if necessary, laws should be enacted to enforce such inspection.

QUESTION 8.—A committee to discuss these matters would be ultimately productive of great good, and as in many other similar cases lead to some understanding as to uniformity of practice. I hope to see this matter taken up with a view of further action.

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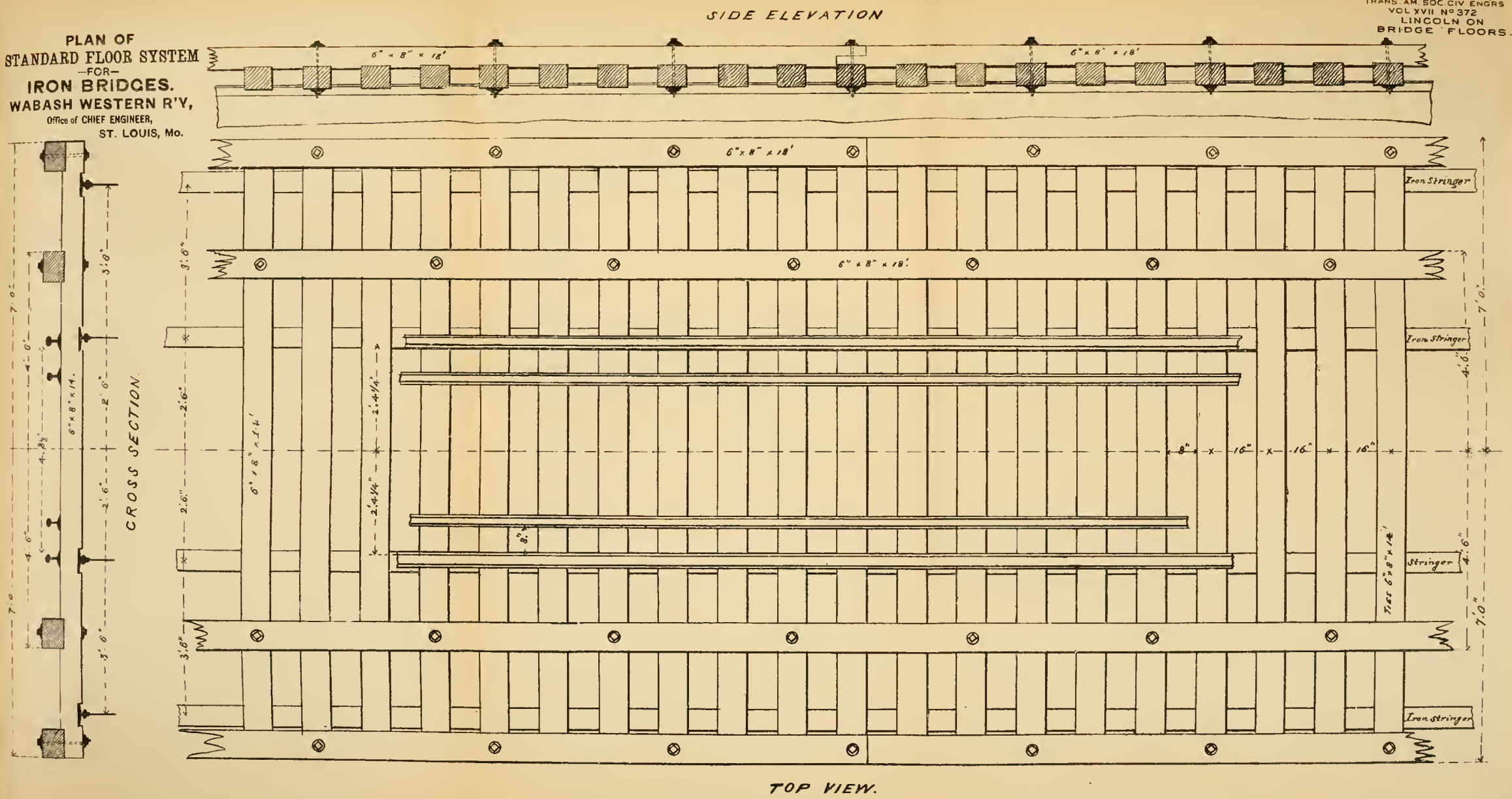
JOSEPH M. WILSON, M. Am. Soc. C. E.—I confess to not being very familiar with the powers possessed by Congress, as compared with those of the States, for framing laws governing matters of proper strength in railroad constructions, but the vast extent of our railroads, spreading all over the country, in utter disregard of State lines, the same road often running through several States in succession, and the applicability of a common law for all, would appear to make it advisable, if possible, that the Bridge Inspection question should be controlled by Congress rather than by State laws.

The common custom and great advantages of running through lines for very long distances and of transporting freight and passengers from one road to another without change of cars or break in bulk; also the tendency to uniformity on the various leading roads in engine service for the several classes of engines, all conduce towards a similarity of rolling loads, and render it advisable, if possible, to adopt a standard specified loading for bridge computations. If, in arranging this, an assumption can be made that would include the effects of both cars and engines, it would lead to simplification.

So many changes are being made from time to time, particularly in engines that if a standard were adopted for the engine loads, conforming to or covering a particular series of engines, it would not remain standard very long, and therefore it would seem best if engines are to be considered separately, to assume a typical engine, designed not only to cover existing types, but to anticipate the outcome of the future. It would seem, however, that one might be able to work up a loading that

PLATE XLI
TRANS. AM. SOC. CIV ENGRS
VOL XVII No 372
LINCOLN ON
BRIDGE FLOORS.

PLAN OF
STANDARD FLOOR SYSTEM
—FOR—
IRON BRIDGES.
WABASH WESTERN R'Y,
Office of CHIEF ENGINEER,
ST. LOUIS, Mo.



would cover both engines and cars, particularly as the present tendency is to make a large margin in loadings, and this margin would probably be sufficient to include not only all variations of locomotives, but the differences between their loads and those from trains. The matter would require more care and study than can be given at this time, but there are certainly possibilities in this direction, and it would be a great point if there was a standard established by law for the designing of all new bridges.

Where old bridges are to be inspected, criticised and accepted or condemned, proper economical considerations would demand that they be considered in reference to the actual service that they have to undergo. Branch roads, with light trains at long intervals, would answer with lighter bridges than a main line with heavy traffic and frequent trains, and an extensive road with branch lines can often replace its light main line bridges by heavier ones, using up the old bridges on the branch lines. I do not mean to have it inferred from this that I favor light bridges on branch lines. By no means. Where a new bridge is to be built for a light branch road, I would always build it up to standard, as in the various changes in the direction of traffic, to which experience can well testify, no one knows how soon a branch line may become subject to heavy service. So far as the framing of a law is concerned to govern this subject, railroads might be classified as say first, second and third class, depending upon the kind of business and loads carried over them, and the operating companies could be prevented by penalty from placing a heavier class of business upon a line than that for which it is registered.

For the designing of new bridges, I am decidedly in favor of a considerably heavier live load than that generally used. I believe I was the first to inaugurate the system of using typical designs representing engines of the future for loading, and my experience with their use shows that it only took about eight years to reach with actual service the advanced limits, which, at the time they were adopted, it was supposed would cover the case for many years to come. This experience justifies me in saying that I think this limit for live load should be made much higher than it generally is. That recently authorized by the Pennsylvania Railroad meets with my approval, and as this road is a very prominent one in extent and position, and the question of coming to some uniform standard is a very important one, I should be in favor of adopting it. It is only an amplification of what I have been endeavoring to work up to for years.

I am not yet converted to the English system of loading small bridges with solid floor and ballast so as to increase the proportion of dead to live load, but I thoroughly believe in making them much stronger than long spans, in proportion as they are smaller, and the live load increases in comparison with the dead load. I would rather reduce the allowable stresses per square inch than increase the dead load.

The adoption of a standard floor system would be a good move if it could be done. But it would be almost impossible to get all engineers to think alike on this matter. I have known those who adopted standards, and frequently found reason to change them afterwards.

I do not like any guard system which provides for guards placed inside of the rails, but much prefer them outside, where they are far less apt to give trouble in case of obstructions falling on the track, and to my mind outside guards can be made equally effective for their purposes as inside ones.

My idea of a properly organized system on a railroad for bridge inspection and maintenance would be something like the following:

There should be a complete tabulated list of all the bridges on the road kept on file in the proper office of the company, that of the chief engineer, the engineer of bridges, or the office representing that branch of the service. If the extent of the road is sufficient to justify it, there may be an engineer of bridges in general charge of this whole department.

This list should give the bridges according to number, first over the main line and then on the several branches from their junction, showing location, kind, size, general dimensions and other particulars.

Then there should be on file in the same office a complete set of drawings for said bridges, showing not only the general plans but complete in every detail, with strain sheets calculated under certain assumed loading, so that a competent engineer may, by examination of these plans, satisfy himself as to the proper design, proportions and strength of each structure, not only as to its general parts, but for every detail. Whenever any changes are made these lists and plans should be kept corrected up to date.

In taking up the bridge question for a road where very little attention has previously been paid to it, one of the first things to do is to prepare these lists and detail plans, and to have full computations made on each bridge, not for an ideal load of the future, which is a question for new bridges, but for the actual loads in use.

With this data before him, the expert engineer can determine how far each bridge is adapted to safely carry the service put upon it, and how far he can allow parts which may be light to fall below the standard limits without condemnation.

He will now proceed to make a close personal inspection of each structure, not only to confirm his theoretical deductions but to examine for faults, decay, breaks, wear and tear or other practical questions which may affect the case more or less seriously.

Each division of the road, such as is covered usually by a division superintendent, say one hundred and fifty miles or more in length, should have on it an assistant engineer, one who is a graduate of a reputable scientific engineering college, who has had some years' actual service on a road under operation, and who has had some experience in bridges.

With him, and under his direction, should be what in the old days of wooden bridges was designated the "master carpenter," but who in these days might have a more appropriate title, such as superintendent of bridges, or something better. This man should be eminently practical. He should understand how to handle and use workmen and mechanics, how to frame and erect wooden bridges, Howe trusses particularly, trestles, false works, how to erect and repair iron bridges, even if it may not be desirable for him to actually undertake the work of erection, and he should know something of the general principles of the action of bridges.

All matters of maintenance would come directly in his charge, under the assistant engineer.

These two men, the assistant engineer and the superintendent of bridges, should work together. On them rests the direct responsibility for the care of the bridges; the daily, weekly or monthly examinations, as may be necessary, are to be made by them. These should be supplemented by the daily inspection of the road foreman, whose duty it should be to go over his subdivision each day, and to examine for defects not only in track but in bridges and everything else on the line. If matters occur that require prompt action they are to attend to them, and to report to higher authority, by which they would always be governed in any expenditures, modifications or changes of structures. If they need further expert examination they can ask for it. They have the direct responsibility, and must take any instant precautions necessary for safety, so that no risks may be run.

After the first detailed examination by an expert, whether a regularly appointed engineer of bridges or only called in for the occasion, together with the preparation of detailed plans, etc., copies of which should always be on file in the office of the assistant engineer of the division, then further inspections by experts may be made only once per year, or, perhaps, say once in two years, with special inspections as may be required, to be made by the engineer of bridges or a specially appointed consulting engineer, but one who necessarily must be an expert in his particular calling. The annual inspection is only an ordinary one, which, of course has grave responsibilities connected with it, but it is not intended to relieve the responsibility of those who have direct charge of the bridges. It should be made in company with the assistant engineer and superintendent of bridges who should, in any examination report freely for the benefit of the expert, all facts concerning the working of the structures under service, and ask for advice where wanted. The inspector, on his part, being familiar previously with all theoretical deductions concerning the bridges, or having them before him, should advise, give instructions and directions, make his own notes and embody the results of his examinations in a report to the proper officer of the road.

After such work is regularly organized and goes on from year to year, each bridge gets to be thoroughly known on its merits, as to its details, its peculiarities and its capabilities. All is under proper control, and repairs, strengthening or renewal will take place in due season as needed, based on the reports made on the structure. If these reports are faithfully made by a competent man, and the advice therein given is conscientiously carried into effect by the railroad company, there is no reason why the traveling public should not be efficiently protected.

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James G. Dagron, M. Am. Soc. C. E.—1. It is, in my opinion, desirable to avoid the interference of the National or State Legislatures in this connection. Any law framed by non-experts could but be deficient and ill-considered, and should such legislation be passed it would probably either add to the powers of the railway commissions in the States where they exist, or create a corps of State Inspectors of Bridges who would be appointed officers. These officials might, in rare cases, be competent men, but there is a strong likelihood of their being appointed more on account of their political influence than of their professional standing and experience.

The remedy lies in the hands of the railway companies themselves, inspection of railway structures is not a new art, and the services of competent and experienced men can be readily obtained.

Every railway company whose mileage is sufficient to warrant it, should have a Superintendent of Bridges on its permanent staff. This official should be an engineer, fully capable of determining the stresses induced in the structures, and thoroughly versed in the practical details of the manufacture of the different parts, and in the characteristics of the material; he should make a thorough inspection of all structures on the line of road at least every six months, and preferably at shorter intervals, reporting their condition to the executive officers. Bridge watchmen, trained by the Superintendent of Bridges as to the points to be noted, should make more frequent examinations of the structures under their charge, reporting at once to him anything amiss discovered by them. In the case of roads whose mileage is not great enough to warrant the permanent employment of an expert, one should be called in at least once every year to make a thorough inspection of their structures, and he should instruct the bridge watchmen as above stated, they in this case reporting directly to the proper officer.

2. Proper bridge inspection consists in a careful and minute examination of all the parts of the structure, both as to their construction and as to their action in service. It should be seen that each part is doing its proper duty and in condition to continue doing so.

All bridges during their construction at the shops should be carefully inspected and the material entering into them properly tested.

This work should be in charge of thoroughly competent men, and not, as is too frequently the case, in the charge of those neither theoretically nor practically qualified for the work.

3. There is no doubt that a revision of specified rolling loads is advisable, as in some cases the actual loads now in use and coming into use exceed the specified loads. It is questionable however whether a standard could be adopted satisfying all conditions.

4. I think that a standard bridge floor could be adopted after full discussion. Such a floor should be strong enough to carry safely a derailed engine or train, the spaces between ties not exceeding four inches. The ties should be notched one-half inch over the stringers, every third tie being fastened to the guard rail and stringer by a $\frac{3}{4}$-inch bolt and intermediate ties fastened to the guard rails by $\frac{5}{8}$-inch bolts, the dimensions of ties to be determined by the distance apart of stringers but to be not less than eight inches deep. Guard rails to be 6 x 8 inches and notched one inch over ties.

5. It would be advantageous in many respects to have bridges of small span made strong enough for a buckle-plate floor, and a continuous coat of ballast on the bridge; such bridges would afford more resistance to impact and vibration, thus adding to their life. This is the general practice in Europe. I am not prepared to state up to what span this should apply.

6. I am heartily in favor of the use of a safety-guard (Latimer, or one equally as good) upon all structures exceeding sixty feet in span.

7. Overhead crossings should be required except when absolutely impracticable, and in this case interlocking apparatus with derailing switches should always be used.

8. Legislation is to be avoided as much as possible, and, if necessary, to be only asked for after complete study of the question by experts and on their recommendation.

9. I would suggest the appointment of a committee, taken from the Society, to thoroughly study the question and to report at the next annual meeting or next annual Convention.

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J. A. L. Waddell, M. Am. Soc. C. E.—1. Let the American Society of Civil Engineers present a memorial to Congress suggesting that laws be passed which will compel every railroad company in the United States to have prepared and filed, either at Washington or at the capital of the State under which the company was chartered, a report by a duly qualified expert upon the strength and capacity of every bridge upon that company's lines.

This report should consist of a diagram showing sections of all main members and the stresses to which they are subjected by the standard

maximum load adopted by the company for the portion of its line upon which the bridge is located, and which load the company under no circumstances should be allowed to exceed.

It should also contain sketches of all details showing clearly the sizes of all parts of each connection, the chord packing, riveting, and all other information that would be necessary for an engineer to enable him to report upon the structure.

In addition to the preceeding it should give the inspector's opinion as to the real capacity of the bridge, considering both the main members and their details, with suggestions as to what changes should be made in order to have the structure comply with legal requirements.

The railroad company should be allowed a reasonable time for making these changes, the length of same depending upon the gravity of the defects; and, in case of non-compliance, proceedings to stop traffic should be undertaken by the Government.

If the railroad company refuse to accept the inspector's report, the matter should be referred to a standing committee, whose action on such matters should be final.

To prepare the laws relating to inspection and capacity of railway structures, a committee might be appointed, thus: The Government to choose a prominent member of the legal profession who has had experience in railroad matters, the president of the American Society of Civil Engineers to appoint two bridge experts, and these three to choose another prominent member of the legal profession and a railroad engineer of great experience and established reputation.

Let the committee make a classification of roads according to the character of their rolling stock and amount of traffic, then prepare general specifications for bridge design so complete as to cover every detail, these specifications to be used for all new bridges, old bridges being allowed a certain percentage or certain percentages of deviation from the requirements.

After the passage of these laws, let the Government give authority to the American Society of Civil Engineers to appoint a board of bridge experts, to acts as examiners of applicants for the position of "qualified bridge inspectors." The members of this board should be engineers who are located all over the country, so as not to require an applicant to travel any further than can be avoided. Any one member might hold an examination (written), the questions being prepared by a committee of the board and sent under seal to the applicant in care of the examiner.

Each member of the board should see the examination papers and mark them, the average of all marks determining the passing or rejection of the candidate.

In case of any three members of the board being able to meet to hold an examination, the latter might be oral and the candidate be passed or rejected solely by these three members.

No one who is not a full member of the American Society of Civil Engineers should be allowed to have an examination; but if this arrangement be adopted, the methods of voting upon candidates in the American Society of Civil Engineers should be modified.

A candidate should be required to have had ten years' practical experience (or, if a graduate of one of certain specified technical schools, six years') including at least three years devoted to bridge work. By following this method none but really competent experts would be appointed.

The Board of Examiners should have the power to, at any time, annul the certificate of any inspector, when proper reason therefor exists.

Maximum charges for inspection should be fixed by the Government, but these limits should be high and based upon a salary of at least one thousand dollars per month. Otherwise competent engineers would not care to practice at bridge inspection.

2. There are two kind of bridge inspection, viz.:

*A.* Inspection of structures whose dimensions are not on record.

*B.* Inspection of structures whose dimensions are on record.

The former is, of course, much more extensive and thorough than the latter. It should be made as follows or in some similar way:

I. Measure systematically the main dimensions of the structure and the sections of all the principal members, recording them always in a certain manner, determined by experience to be the best, so that any particular data may be found immediately by inspecting the field notes, which, by the way, should be made in ink.

II. Measure and record systematically the sizes of all parts in the neighborhood of each panel point and each connection of main members, showing number, spacing and diameter of rivets, the packing, including the distance of center line of each piece from plane of symmetry, dimensions of eye-bar heads, thickness of bearings, and, in short, every dimension that could under any circumstances be required.

III. Measure and record systematically all the details of main members between panel points or connections, for instance, sizes of lacing bars, stay plates, stiffening angles, etc.

IV. Examine the structure carefully to find any faults in manufacture or design, such as loose or unequally strained tension members, bad packing, omission of fillers, bad riveting, twisted or otherwise distorted members, inefficient bracing, loose connections, etc., also the effect of wear such as loose rivets, bent pins, rust, decayed timber, cracked castings and defective masonry or other material at pedestals.

V. Look to the efficiency of the floor system proper, viz.: the ties, rails and guards, also to the means of protecting structure from injury by fire, derailment, vibration, etc.

VI. Examine thoroughly and make notes upon the substructure, giving the principal measurements, quality and condition of materials,

etc. Describe the crossing of the stream or chasm, noting if possible, high and low water, velocity of stream, and any other information that may be of use.

VII. Note the effect upon the bridge of rapidly passing trains, measuring and recording, if thought necessary, the deflections.

VIII. Note if possible names of designer and manufacturer and date of erection.

IX. Record in note-book the names of the members of the inspecting party, the date and the time spent in making measurements.

The inspection of structures whose dimensions are on record should be made simply with the view of ascertaining the effect of wear upon the structure.

The items are mentioned under the previous headings numbered IV, V, VI and VII. Before making such an inspection the inspector should read carefully the notes of the previous inspections, and determine where to look specially for the effects of wear.

3. There should be not one but a number of standards for rolling loads, graded according to the train weight and the present and probable future traffic.

It would be a great waste of money to require the small western roads to adopt bridges calculated to carry the heaviest traffic in the country. It is no serious matter for a railroad company to remove and replace a structure that is too light for an increased traffic; for the light bridge can generally be used on one of their branch lines, or, if well designed, be sold to some other road.

For proportioning the floor system and primary truss members the amount and distribution of weight of one or more standard typical engines should be used, but the chord stresses of trusses should be calculated for a uniformly distributed load, and the web stresses of trusses for another uniform but advancing load headed, if thought necessary, by an engine excess and even followed by another engine excess, if so much refinement of calculations be considered advisable.

The second engine excess could be assumed at the nearest panel point.

The ultra refinement of calculating the effect of engine load concentrations upon main members of trusses can be fully appreciated only by the computers of bridge companies whose time is utterly wasted in this manner.

It seems hardly necessary to mention that the equivalent uniform loads for both chords and webs should vary with the length of span.

4. It is, indeed, most expedient to adopt a standard bridge floor. The requirements for same should be :

I.—That it carry safely across the structure a derailed locomotive or train without moving out of position a single tie.

II.—That it be designed to prevent the starting of vibrations that would be prejudicial to the structure.

III.—That it provide a place of refuge from passing trains for persons on the bridge.

IV.—That its cost be made a minimum.

To accomplish the above the writer would recommend that the stringers be placed directly under the rails, that the ties be of 6 x 8-inch timbers laid on flat and spaced twelve inches centers, that every sixth or seventh tie extend over the whole clear width of structure so as to support at each end a longitudinal 3 x 12-inch plank for a foot walk, the remaining ties not exceeding seven feet in length; that every tie be bolted at each end to the stringers or to wooden shims resting thereon, and which are rigidly bolted to same; and that no outer guard rails be used, but instead inner guard rails of angle-irons placed about six inches clear of the rails and bolted to alternate ties, these guards to extend on to the embankment, where they should be drawn gradually together and terminated by an acute-angled frog.

5. Yes. The limit of pin-connected spans should be not less than one hundred feet, but lattice girders, as ordinarily built, should be ruled out, and single intersection triangular riveted girders used instead, great care being paid to the proportioning of details, and metal and rivets being used liberally in the design.

6. By all means, and the limiting width should be small.

7. Yes, in districts at all thickly populated; but not for some time to come upon the roads of the far West.

8. Legislation as to all these points is not only expedient but is fast becoming an absolute necessity. The condition of many bridges, especially in the West, is something frightful. Any man has the right to call himself a bridge builder and to help fill the country with man-traps of the worst description.

There are many bridges in the West in which the iron is every day over-strained from fifty to one hundred per cent.—this under the assumption that the live loads are applied without impact, while in reality the structures are so loose-jointed that the vibrations induced thereby must cause the dynamic effect to increase the calculated stresses by at least fifty per cent.

The difficulty is and has been that railroad managers and chief engineers, capable men as they generally are, are entirely incompetent to pass judgment on the merits of bridge designs submitted for their approval, so they generally let the work to the lowest bidder and thus accept in many cases the most inefficient design.

They have a notion that they are possessed of very sound practical ideas about bridges, while, in truth, they are ignorant of the first principles of design.

The only safe method for such men to adopt is to employ the services of an expert in bridge designing, and the only safe method for the country in general is to pass a law compelling all railroad bridge companies to submit both diagrams of stresses and working drawings for approval by either a State Bridge Engineer or a board of engineers legally appointed.

9. There is no method which will have a more beneficial effect upon the quality of future bridges than that of organizing an association of bridge designers and builders for the purpose of discussing bridge designing in all its details with the ultimate object of adopting standard bridge specifications so complete that upon looking over a design any competent engineer can determine, without any doubt, whether the proposed structure be fully up to the specifications in the most minute particular. Such an idea is by no means Utopian, although it will no doubt be characterized as such by many engineers.

It might take several years to arrive at the object of such an organization, but if the latter were composed of the right kind of men, and if a committee of them were well paid by the Government for their time, and permitted to make experiments upon bridge material with Government money, there would be no doubt about its success. If the Government is willing to spend millions upon a study of the physics and hydraulics of the great rivers, why should it not be willing to devote a million or even half a million of dollars to the investigation of how to build first-class bridges at the minimum expense?

Can any engineer have any doubt about the value of such experiments, if made systematically and with a practical object always in view when making each experiment?

At the meeting of such an organization every word spoken should be recorded, printed and distributed for further comment. This would result in obtaining the views of engineers not in the association, and in insuring to every man in the profession an opportunity to present his opinions to the public, if said opinions were worthy of such presentation.

Why cannot such an organization be formed under the auspices of the American Society of Civil Engineers?

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E. S. Philbrick, M. Am. Soc. C. E.—I have hurriedly put down something in reply to the circular. It is based upon our experience in the State of Massachusetts, where legislation through a railroad commission has done much, we think, towards an enlightened guidance of railway administration on such points as regards the public safety. The cardinal principle with us is publicity in all facts, and the strict accountability of every corporation to an enlightened public opinion. Our commission has done much, we think, too, towards educating and creating such a public opinion, which, in a democratic community is a mighty sovereign.

1. Railways should be required by law to file plans and specifications describing each of their bridges in detail, with such uniformity as can be secured, in the office of a State Board of Commissioners who should be authorized to employ experts to examine and report upon the sufficiency of such plans, etc.

An annual inspection and report should be made upon every bridge by an inspector in the employ of the owner of such bridge.

2. Proper bridge inspection should include:

(*a.*) An examination of the strain sheet to see if it is based upon proper loads, and that it is computed to show the maximum strains.

(*b.*) An examination of the dimensions of every member to ascertain the actual unit strains by comparing with strain sheet.

(*c.*) A search for signs of depreciation and wear, such as loose rivets in riveted members, excessive deflection under rolling loads, inefficient lateral bracing, loose bolts or screws, rust. Hidden parts, not accessible to paint or to vision should be condemned.

(*d.*) Change of form in any combination or single member.

3. Prudence requires us to anticipate the future possible and probable increase of the weight of rolling stock by using heavier standards than those in actual use upon any railway in question. But it may not be proper to apply the same standard indiscriminately to all lines. Regard should be had to the probable nature of the traffic and whether or not it is connected with other lines from which traffic is received from remote parts or whether it is comparatively isolated and independent of foreign rolling stock. For all trunk lines which look for a large traffic from considerable distances a standard rolling load and wheel load with axle distances for locomotives could and should be adopted.

4. Standard should be adopted for bridge floors, varying somewhat according to the volume and nature of the traffic.

5. Bridges with buckle-plate floors and continuous ballast afford the advantage of a more smooth and noiseless track and greater security in case of derailment, but they are open to the serious defect of concealed rust, and are, in the writer's opinion, not to be recommended for this reason.

6. Some safety guard should be applied at all openings in the road bed over ten feet in length.

7. Level crossings should never be allowed for tracks used by different administrations, unless protected by interlocking signals and derailing apparatus or its equivalent.

8. Legislation is expedient if so arranged and guarded as to never relieve the railway company from its responsibility in case of neglect or break down. It is preferable to have a State Board of Commis-

sioners, whose duty it should be to see that all railways within each State conform to statutes and to all reasonable safeguards. Publicity through such a board can effect much reform, though their efficiency must largely depend upon the earnestness, tact and industry of the members.

9. It may not be so easy to induce any considerable number of railway superintendents to meet and discuss such subjects as in the case of manufacturers of rolling stock, but if they could be so induced, much good would result by the interchange of ideas and the broadening of views among men who are tempted to become routinists from living in too narrow a circle.

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Andrew Bryson, M. Am. Soc. C. E.—In reply to your circular relating to the inspection and maintainance of railway structures, the following is submitted, embodying in as concise a manner as possible, some of the writer's ideas concerning the questions therein asked.

1. What measure, legal or other, can be taken to insure a proper inspection of railway bridges ?

Compel, by act of Legislature, every railway company to either keep in their employ competent men to inspect their bridges, or to have an inspection made at least once a year by some competent outside engineer ; also furnish strain sheets and detail drawings to the Railroad Commissioners, who should have a sufficient force of experts in their employ to verify the proportioning of parts, and, if deemed necessary, verify also the structures as built and see that they correspond with the drawings and calculations. A printed form for inspection reports should be prescribed to be filled out by every inspector, for each structure examined, showing in minute detail, when and how the examination was made. These reports to be made in triplicate. One copy to be retained by the examining engineer, one to go to the railway company and the other to be sent, by the inspector, to the Railroad Commissioners, each one to be signed by the inspector, with date and post-office address subjoined.

2. What is proper bridge inspection ?

It consists of two separate and distinct examinations, one purely theoretical and the other practical ; either one alone is little, if any, better than none at all. 1st, the strain sheet must be verified ; 2d, the proportioning of parts to meet the requirements determined by the specifications and strain sheet ; and 3d, an actual detailed examination of the structure from bed plates to ties and rails. On roads running frequent and heavy trains, one or more members subjected to sudden shocks, like floor-beam hangers, should be taken out, replaced by new, and tested to destruction, with a careful record kept in the

"Bridge Book." One or more of these small, easily removed members from each bridge, should be tested every year. Short riveted truss bridges should have a member of the truss taken out, replaced by a new one, and tested, once in every five (5) or ten (10) years, depending on the amount of traffic passing over it. Once in eight (8) to twelve (12) years (depending on the amount of traffic) all truss bridges under, say 150 feet span, should have a truss member removed and tested.

3. Should there not be a standard specified rolling load much heavier than as now used and a specified wheel base for rolling loads?

I think more than one standard should be prescribed to correspond with the nature of the traffic and importance of the line. It would be something of a hardship and waste to force a road running five or six trains a day, with thirty (30) ton engines, with no prospective increase either in number of trains, or weight of engines, to build such structures as would be required for another road running a great number of trains and seventy (70) or eighty (80) ton engines.

4. Is it not expedient to adopt a standard bridge floor ?

It seems to me that if the width and strength of floor were specified the manner of meeting these requirements might be left to the individual designers. Guard timbers should be prescribed, one style high, for southern roads, another low for northern roads. Where snow-plows are used, they are about thirteen feet wide, having the plow proper as close to the rail as possible ; consequently high guards near the rails cannot be used. In the writer's opinion, however, an iron guard railing in addition to the ordinary track guards, should be used, high enough to bear against the body of a car in the event of derailment. This guard railing could be easily braced outside, by an extension of the floor beams, and extended beyond the ends of the bridge, gradually widening so as to form a perfect protection to the structure ; they could be placed wide enough apart to pass snow-plows, and yet the angle that a car could take would be slight, so there would be no possibility of going through the truss, or if a deck bridge, of going over the side. With ties laid close enough to carry a derailed truck as should certainly be the case, such accidents as have occurred during the past year, in one or two instances, would be rendered well-nigh impossible. Greased timber wales might be attached to this guard railing and some spring action provided, similar to ferry slip guard piling, to take up any possible shocks.

5. Should not bridges of small span be made strong enough for a buckle-plate floor, etc. ?

Yes ; but I am not prepared to say up to what span. To 6 and 7 I would say yes, in capital letters.

As to question 8, legislation seems to me to be desirable as being probably the surest way of enforcing necessary safeguards ; it is, however, a technical question and the proper requirements can only be

properly specified by technical men. To that end it would seem eminently proper for the leading engineering society of the country to adopt a series of "suggestions" which should by act of legislature, become specifications, with, at the same time, power given to the Railroad Commissioners to enforce the same; penalties for non-compliance with the act to also be incorporated with it.

Bridge inspections are altogether too liable to be made in a purely perfunctory manner. Sometimes those in authority over the engineer are exceedingly chary of granting requisitions for repairs of structures; if weakness is suspected no aid is given to prove the case one way or the other, but arguments are brought forward to disabuse the engineer's mind of its "scare;" therefore an engineer should not only be allowed to make his examination as carefully as possible, even to taking out members for test, but he should be compelled by law to do so. When railway managers find their engineer is obliged to report the exact condition of each structure in writing, over his own signature, not only to the company, but also to the State, they will know in case of accident just where the blame should lie, and engineers with rather weak spinal columns, afraid to come out boldly and say "this is unsafe, I will not be responsible for it," will find their vertebræ very materially strengthened by such a law.

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Robert A. Shailer, M. Am. Soc. C. E.—My experience has taught me that the many damaging abuses to the parts comprising an iron or steel bridge, such as severe drifting of holes, bending plates cold, etc., are far more likely to arise during the erection of a structure than while the same is being fabricated in any of our reputable bridge works; and I therefore desire to emphasize the fact, that not only in the shops and after erection is inspection necessary, but also during the process of erection.

There seems to be no inspector present to speak for that useful branch of our profession, so in their cause I will remark that inspectors are but human beings, and therefore cannot be relied upon to be in more than one place at a time, or to see things not visible to the naked eye.

Granted that we have a good and faithful inspector, those of us who design bridges must not try to sail too close to the wind, so to speak, in the matter of details, but must be sure to have them strong enough, especially in all field connections, even if the bearing surfaces are not perfect, or there are a few loose rivets, or any other defects creep in which the inspector has not discovered or has been unable to prevent or rectify.

Guard rails on a bridge are useful for two purposes: first, to keep derailed wheels close to the rail and thus prevent the trucks from slewing round; and second, to keep the ties from bunching when a derailed train is pounding over them.

The distance between the edge of the guard and track rails should be just sufficient to allow a driving wheel of a locomotive to drop down between the two said rails. Ordinary old rails placed, say 10 inches inside of track rails, and carried two or three hundred feet beyond the ends of the truss bridges, and there brought together to a point at the center of track, will fulfill the first purpose of a guard rail admirably; and a 10 x 10 or 12 x 12 inch timber notched over the ties at or near their ends, will prevent the latter bunching, and if said timbers are well bolted will serve as an extra safety stringer. 6 x 8 strips notched into the ties and placed 10 inches outside of the track rails will be cheaper than above and will answer both purposes.

If the trucks of a car get slewed, so as to cause the wheels to mount and jump over guard rails placed as above, nothing short of a miracle will save the truss from being wrecked.

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W. A. Haven, M. Am. Soc. C. E.—The inspection and maintenance of railroad structures is a subject on which I have maintained opinions, and I should have liked to be present at the discussion by the Convention, or to have sent you some of my views in writing, but you know I am not yet able to sit up long enough to write a page. These are very important matters, and I hope a good committee will be appointed to get the opinions of the members of the Society. I will try to send them my ideas. I will only in this place say that the American Society of Civil Engineers, should not allow that there is a standard of goodness (except the Anglo-Saxon's weights and measures), and the very idea of uniformity in railroad structures is a step backward.

One thing however I would say, viz.: Suggestion No. 6 is good and there are some contrivances equally as good, say the Latimer guard combined with the Erie Railroad shunt system, and posts at end of bridges, should be recommended for all bridges over 30-feet span.

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J. M. Goodwin, M. Am. Soc. C. E.—Any inspection of a bridge, or like structure, made by a person of fair intelligence and capacity for observation, intending to ascertain the existing facts in the matter of the physical condition of such structure, with the purpose of reporting the results of his inspection, will be measurably useful ; now and again an inspection, merely superficial, will opportunely discover conditions, a continuance of which even for a few hours beyond the moment of observation would result in disaster. And one may properly agree that perfunctory "inspections," made by employees of a railway company or by State officials or their employees, have some elements of value. But a periodic inspection of the bridges of a railway, or of the railway bridges of a State, as such operation is ordinarily performed, is hardly

anything more than a ceremonious observance of some loosely constructed mandatory clause of a statute. And such "inspection" may be worse than useless in its results, as it would be in any case where an inspector should fail to condemn an insecure bridge.

The Ashtabula Bridge was "inspected" periodically from the time of its erection up to the time of its collapse. The inspector was thorough in his examination: he regularly went through the bridge (double-track deck bridge), and saw every part of it; but at every such inspection he looked with unappreciating eyes at conditions, the sight of which, had he possessed the knowledge of a bridge specialist of the present day, would have induced him to first get himself carefully out of the bridge, and to immediately thereafter cause suspension of all traffic over the structure.

This inspector regularly reported the bridge "all right," or to speak more exactly did not report it as in any way "wrong;" and the railway company, relying on the care and intelligence of a man of proved faithfulness, and general efficiency as a boss carpenter, had no doubts as to the sufficiency of the bridge. Now the inspection performed in this case was quite critical in comparison with that ordinarily given, even at this late day, by persons going through the form of inspection to satisfy the letter of some "law."

Doubtless in some cases, and perhaps in many cases, the person employed by a State Board to inspect the railway bridges of its State, is entirely competent to conduct a proper inspection of any ordinary railway bridge; but under current practice merely cursory or superficial inspections are in such cases made. The operation of looking at a bridge from the rear platform of a car running over such bridge, may be held to be a "cursory" inspection; and any inspection which does not go beyond looking at a thing is "superficial" according to the intent of my nomenclature.

In answer to the question "What is proper bridge inspection?" one may broadly say it is an inspection through which the elements, material and structural, and the physical condition of the bridge inspected are developed and set forth, by description and diagram respectively, in a report explicitly and exactly treating each and every fact and detail of consequence in the matters under examination. For a plan or scheme according to which such inspection may best be made we should apply to the bridge specialists as best qualified to supply such plan. But let the plan include the making of a plain statement, in the nature of a deduction from the ascertained facts in the case, wherein the inspector shall, under pledge of his honor and professional reputation, judge the bridge; and shall say, explicitly, all that in view of the ascertained facts he ought to say.

If the several societies of civil engineers of America will, by a congress of delegates, instructed for the purpose, formulate a scheme for

inspection of railway bridges; and if the profession, as a body, will thereafter discountenance the employment, by railroad companies or by State authorities, for the making of inspections, of persons not engineers of standing and proved ability, proper inspections of railway bridges will quite promptly follow.

But no inspection of a railway bridge made by a person who will be, however remotely, adversely affected by a condemnation or unfavorable criticism of the structure, will be accepted by the public as a thoroughly "proper" inspection. Hence, inspections of the kind herein considered, *i. e.*: those made for the purpose of securing proper construction and maintenance of public highways, are properly to be made by judiciously selected agents of the public.

Question 3 of the circular is hardly to be answered by a bare "yes" or "no;" nor could one venture to answer except after a statement of his view of the several and various existing conditions necessarily to be considered in order to the forming of an intelligent opinion in the matter. Hence, as time does not serve, I pass this query.

In regard to bridge floors I am of opinion that every railway bridge should be floored so that a locomotive may smoothly travel on the floor, from end to end of the bridge; such floor need be no more than two feet wider than the gauge of the track; it should have a high and very stout guard rail (angle-iron on timber, or an equivalent arrangement) on each edge of it. These guard rails should extend for some distance outward from each end of the bridge, flaring out from the rail so as to catch and lead toward the rail any derailed wheel encountering either of them.

Every "opening," however narrow, should be provided with similar safeguards. With these in use the suggested "continuous coat of ballast" carried by "plate floor," across minor openings, will be unnecessary.

Question 6. The "Latimer" replacer has in several notable instances, of record, saved trains, and bridges, from wreck. This very effective apparatus for replacing on the rail derailed wheels, should be set in each approach to every bridge.

Question 7. Where one railway crosses another at grade means should be used for positively preventing "crossing-collisions." Use of "interlocking" apparatus, in connection with derailing, or diverting switches, is certainly effective in this regard, and should be universal.

Question 8. As to legislation as a means for bringing about desirable reforms and improvements in railway practice and appliances: The instances in which legislation, had with the purpose indicated, has worked more harm than good, are very much more numerous than those in which the opposite effect has been noted. Guided by common law, courts decree that railroads must employ, in the service of the public,

of all devices known, those which have approved themselves to be the best; failing to do that they must make good all damage sustained in consequence of such failure, as far as money can make such damage good. A statute might say that a certain contrivance is better than any other of its class. But that would not bar evidence, on occasion, showing that such contrivance is not as good as some other well-known appliance. The statute cannot hold against the common law.

I see no necessity for special legislation in behalf of the objects under consideration.

Question 9. I think that the Society will do well to cause the appointment of a committee of its members to prepare a set of resolutions intended to serve as a presentation of the sense of the Society in the matter of inspection of railway bridges. These resolutions to be submitted to members, by circular, with request for comments and suggestions. The committee then to revise the resolutions, and, if cause appear, amend them. The adoption of the resolutions to be then put to vote by letter-ballot; and if a majority of members be in favor of adoption, the resolutions adopted to be declared as setting forth the sense of the Society. Following this the resolutions would be submitted to sister societies for concurrent action. If this be secured, the co-operating societies to proceed in the inspection affair together. Otherwise the American Society to stand to the front alone.

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Albert Lucius, M. Am. Soc. C. E.—Herewith I beg you to find briefly my answers to the questions contained in the circular headed "Suggestions," as follows, viz.:

1. Hold the railway companies strictly accountable for accidents caused by failure of railway bridges on their lines.

2. I hold proper bridge inspection to consist in the examination of all parts of a bridge with the thorough knowledge of the stresses which each member of the bridge has to sustain. I would consider it necessary to have a correct strain sheet of the bridge before examination and to weed out all overstrained details and members even though they do not yet show a positive defect.

3. The specification of stresses to be borne by the various members and the rolling loads constituting the loading of a bridge must be considered together. If the various members of a bridge are proportioned by the stresses prescribed in the "Erie specifications," or specifications based upon it. the train loads specified therein I hold to be sufficient if they are preceded by a heavy load consisting of one consolidation engine and the latter preceded by a yard engine of 48 tons on a wheel base of 12 feet. This would insure heavy bridges for small openings on which the influence of the impact of the moving load is greatest, and it

would insure heavy floors, floor details and counters of long span bridges without increasing the chord members of long spans unnecessarily.

4. I would consider it expedient only to adopt a general diagram establishing general proportions, but not bind all details by fixed rules.

5. I would prefer a floor of closely laid iron ties on iron stringers, the latter spaced sufficiently far apart to give the floor a certain small amount of elasticity, to a floor made of buckle plates and ballasted as I think the latter retains moisture too long, assists corrosion and makes inspection and repairs more difficult.

6. I would favor the use of Latimer guards on all through bridges and on all trestle works and deck bridges with intermediate and battering pins which would be broken down by a train falling over the sides. Their use on other deck bridges would not be material to the safety of the bridge, but as regards the safety of the train I would favor their use on all deck bridges, their approaches and on embankments especially if the latter are on curve. All deck-bridge floors should be of same width as embankments. The end posts of through bridges might be additionally protected by posts planted in front of the end posts and braced back on the bed plates of the bridges. This might prevent some accidents caused by shifted loads or a trailing car, in other cases though, it would probably not prevent a wreck.

7. There is such a wide range in the importance and location of crossings that I would not know how to cover the whole case by one specification. I would consider it highly desirable, though, to restrict the multiplication of grade crossings both of highways and railways, and also to offer legislative inducements to reduce the number of the present grade crossings according to the importance of the case. By legislative inducements in this connection, I mean that negatively—stoppage of trains or reduced speed over grade crossings, might be enforced.

8. I would answer the same as 1.

9. I believe that the result of the deliberations of a large committee selected to prepare a report on the subject matter of the above suggestions of your circular would be beneficent, especially if it confined its labors to the preparation of a general standard specification both as regards loads and stresses by which bridges and their members shall be proportioned without prescribing too closely either the form of structure or the limits to which different forms and styles of girders or trusses shall be used in order to leave some room to the skill and individuality of the designer.

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E. P. Dawley, M. Am. Soc. C. E.—The subject of inspection of railway bridges is of great interest to all of us, and especially to such as are responsible for their care. Answering circular headed "Suggestions," the writer would present opinions about as follows, but while expressing

such opinions, would say that he should consider it a very valuable contribution towards the welfare of the general public and a great aid to engineers of maintenance of way on our various railroads if our Society would put forward with their recommendation some standard for rolling load, and also agree generally upon preferred types of bridges, and further, a general outline of rules to be observed in maintenance of railway bridges.

This would be likely to result in a certain general standard of excellence in these respects which public opinion would cause most railroads to accept, and at same time individuals would be at liberty to exceed such standards of strength and methods of cars to any extent that might seem desirable to themselves.

1. That strain sheets of all bridges, railway, highway and others, be required to be filed with proper State authority (railroad commission if that should seem best). If railroad commission is not made the depository for everything of this kind, then a duplicate set of such bridges as are maintained by railroad companies to be deposited with railroad commission. Authority to be given to railroad commission to cause bridges to be strengthened if they deem it necessary.

But I do not think that legislative measures should go so far as to tend to make railroad officials look for or wait for, notice of defects from any State or other authority. In other words, the State supervision should always remain quite general in its nature, and the direct responsibility be, and remain with the railroad companies, where it belongs.

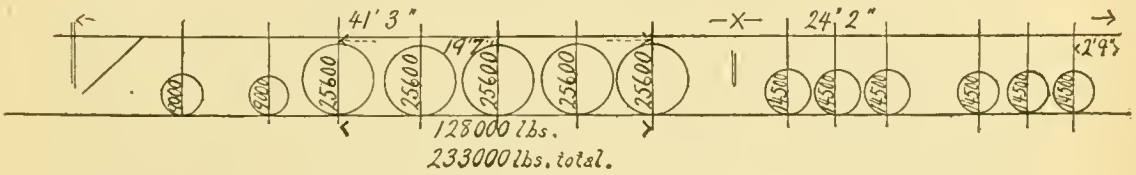
2. A proper inspection of bridges originally first class and at present (so far as strain sheet and sections go) ample for their loads would seem to the writer to be covered by the following—a thorough and minute inspection of every part and piece of the bridge to ascertain, as far as may be done by the eye and the hand and by the ear, the condition and adjustment of each piece in the bridge. To include a very careful search for loose rivets.

Measurements of vertical and lateral deflection and measurements to detect any movement of the bridge on its foundations might be included. Such an inspection to be made once in four months. Between times, an inspection more or less irregular to be made not more than a month apart, sufficiently close to know that everything is in place and in good order so far as can be readily seen. The more important parts at such a time, of course, receiving more careful consideration. The first inspection described might be illustrated by saying that it would probably consume one-quarter to one-half a day or even a day for, say a double track through bridge of 100 to 150 feet span, for engineer and assistant.

The more casual inspection might take an hour.

3. It is doubtless pretty well agreed among engineers that 3 000 pounds per lineal foot of track is not sufficiently heavy for spans 100 feet and less; that is, not sufficiently heavy to discount future service.

This railroad—the New York, Providence and Boston—has used for several years past for rolling load a wheel diagram like cut below followed by a rolling load of 3,000 pounds per lineal foot of each track, the dead load for floor being 500 pounds per lineal foot of each track.




I think it simpler, however, and full as good, say for plate girders of 75 feet span and under, to use 5,000 pounds per lineal foot of each track for live load.

Strains allowed about the same as Erie specifications.

4. About standard floor. It is to be presumed that most roads already have a standard floor of their own, and so long as they are good floors I see no great benefit to be derived from having bridge floors on different roads just alike. The writer considers the following to be a good floor, and uses it for standard: Hard pine timbers 8 x 9 inches and 26 feet long, supported below, in case of a through bridge, by seven stringers, for double track, timbers being laid flatwise, 15 inches center to center.

A guard timber, 8 x 9 inches, outside of each track, and a third one half-way between tracks which are 13 feet centers.

Guard rails running to a point in center of each track are used and continued across the bridge 7 inches from main rail. Floor timbers to be carried out on to the ground, and guard timbers to be made flaring at ends of bridge, as per Child's arrangement for the Latimer safety guard.

5. Am not yet worked up to the idea of buckle-plate floor, as it takes up height (in many places so valuable), and increases difficulty of inspection, etc.

6. Don't know much about the Latimer safety guards, and rather object to rails being continued across bridges only 2 or 3 inches away from main rail; but think the principle a good one of using the guard rails running to a point, and have so used them for some years.

7. Should be one of the two, and the overhead crossing is far ahead of the interlocking apparatus.

8. Think that legislation enforcing, say at least the standard of strength and methods of inspection which might be put forward by a society like the American Society of Civil Engineers, would be a good thing for all parties.

I believe that overhead crossings or the interlocking apparatus should be strictly enforced by law.

I have written at some length and hope you will hear similarly from others. It is the writer's conviction that in a few years from present time our largest ideas of what is required for strength and safety will be considered meager and illiberal for increasing service.

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C. A. MARSHALL, M. Am. Soc. C. E.—Relating to inspection and maintenance of railway structures, I have the following to offer in response to some of the suggestions forwarded:

1. First, legislation requisite is national. The primary inspection of materials and workmanship is of the greatest importance in any individual case, and should be compulsory, and, in some few particulars, standard for the whole country. Design and execution should remain in the hands of engineers and builders as now.

There should be a commission of engineers and experts in building materials appointed annually with some holding over members, for the purpose of examining inspectors and issuing licenses, or these duties might be well incorporated with those of the proposed commission on structural materials. The duties of inspectors to be defined by this commission in accordance with legislation. The commission to receive compensation and to sit for examinations simultaneously in several large cities by division of members.

There should be licenses of three different grades:

*First.*—As to materials.

*Second.*—As to fabrication.

*Third.*—As to compliance with general strength and safety requirements (apart from materials and shop workmanship) which may be set by the commission.

An inspector certifying to any of above three kinds of inspection must make oath that he is not in the pay of, nor pecuniarily interested with, case 1st. The manufacturers or sellers of the material; case 2d. The builders or contractors; case 3d. Either or any of them.

Engineers may become inspectors, but may not deputize the work to unlicensed men; the form of oath should prevent this. It would probably be expedient to arrange for licenses of the third class being taken out by practicing engineers upon satisfactory evidence of competency other than personal examination.

There should be stated maximum fees for inspection of different classes of work. A licensed inspector in the nearest town may not refuse to take work at the regular price—plus mileage, but may take work wherever he can get it, and at lower prices, if he choose. The number of inspectors in a given territory should not be limited, but each must designate some locality, as headquarters.

Inspectors should pay a moderate annual license fee, and it should be made a penal offense after a certain lapse of time to build certain

classes of structures without securing the services of a licensed inspector or inspectors. This the general government can control as well as it now does steamboat boilers, and the inspectors themselves will be active to see that all cases are reported.

There should be a salaried chief inspector, having power to suspend, and two or three deputies to assist him in settling disputes and enforcing regulations, especially the requirement of personal attention to the work.

Builders to be required by law to furnish the inspector, free of charge, besides the usual information, a complete copy of strain sheets and detail drawings, giving full and exact information of all bridge work inspected by him, for which he shall furnish receipt in duplicate, and he shall certify that they correctly represent the work inspected, and further that the requirements of the commission as to workmanship have been complied with. Inspector also to certify under a copy of the specifications for material that they are complied with, including also any requirements of the commission, and give particulars as to where, when and by whom made.

Certificate of an inspector of the third class as to strength and safety shall be obtained by the owner of the structure within a certain time after completion. All the information and certificates to be lodged with a designated officer of the State in which structure is located, and for their own protection the builders shall lodge an inspector's receipt with the chief inspector.

To the State must be left the inspection of completed structures. They may give the work to a Department of Public Safety or a Commission, as seems wise. There is no question but the work should be done and the States should be urged by the National Commission and by all possible means to provide for efficient record-keeping and inspection of structures located within their borders. They should designate an officer to receive the reports of the licensed inspectors of new structures and should provide a sufficient force of experts to thoroughly and systematically inspect. The force needed would be greatest at the start, and much less when good records had been secured and bad structures weeded out.

The National Commission should provide a system of registration and grading of condition of structures including recommended forms of blanks, but must confine themselves to recommendations in this branch of the subject having due regard to work already begun.

2. Proper bridge inspection presupposes good records either at hand or to be made at the time. It should be systematic, but no amount of system can take the place of actual personal inspection by a competent expert in both theory and practice of bridge building.

3. The question of rolling load is overshadowed by that of impact effect. It would be idle to specify heavy loads without at the same time

limiting unit stresses. Think that floor system should be designed for heavier loads than now, and that it would be proper subject for recommendation by committee, but not for legislation for some time yet.

4, 5 and 6. A standard minimum actual weight of floor system would be a good thing on all bridges of less than 150 feet-span, which from 150 feet to 75 feet need not be so heavy as the buckle plate and ballast, below 75 feet should be as heavy as that. Both inside safety guards and outside guards should be required by law; also standard for opening between ties at, say, 4 or 5 inches. In case of deck plate, web or solid girder bridges only, the whole bridge might be considered as floor system. Inside safety guards when bridge is approached on curve should be repeated or extended well back on the curve. I once investigated a fallen truss bridge which proved to have been knocked down by a blow of the corner of a freight car on the end post; two cars had been off the track some two hundred yards back on a curve without the engineer's knowledge. The engine and two cars crossed the span in safety, and the remainder of the train was wrecked. Safety guards terminating near end of bridge would not have saved it, though my remembrance is, none were used. The inside wheel of the car was nearer the outer rail than the inner one when it struck the bridge.

8. I regard the plan detailed in (1) as most expedient and pressing in order to further uniformity and safety in these matters. The dangers to the average bridge of the country are perhaps as great from faulty design as from trains off the track, while to a well-designed bridge the latter constitutes, by far, the chief source of danger. To weed out the weak existing bridges the legislation should be by the State. To guard all bridges existing and to be built against trains off the track, as far as possible to do so, is also in the province of the State, but would be better done in accordance with a standard mode. To set a few necessary standards in this and other particulars should be the work of a national commission, and to bring about such a commission, as well as to influence corporations in the right direction, is a proper task for this Society.

## DISCUSSION AT CONVENTION.

D. J. WHITTEMORE, Past President Am. Soc. C. E.—Mr. Chairman and Members of the American Society of Civil Engineers: Twenty-two years ago I was appointed chief engineer of a company having a line of railway which was then 275 miles long. The company was then in nearly a bankrupt condition, and through decay and use the greater portion of its structures of a perishable nature had to be replaced, and it was a difficult matter to determine how to plan the work of repairs and renewals, and have the expense of the same within the means of the company with due regard to the safety of the public. This, however, was accomplished, and within the last twenty-two years this company has built 2 700 miles of road and acquired by purchase other lines, which bring its mileage up to 5 400 miles at this date. This rapid increase of mileage, involving, as it did, great responsibility on the part of the engineer in the inspection of bridges, impressed upon him the necessity of formulating a complete system, so far as he could devise it, of tests and inspection. The magnitude of the work at once suggested the making and adopting of standard plans of structures. These were made, and after conforming to them for about one year it was found necessary to change nearly all to meet the requirements of increased rolling loads, and I may say that this process has continued and probably will continue to the end. I find that on the system of railway with which I am connected, two per cent. of its length is covered by bridges consisting of spans of from 10 to 400 feet, and generalizing from this I conclude that there are some 2 800 miles of bridging on the 140 000 miles of railway now in this country. It appears to me that the members of this Society cannot do a better service to our profession than to recommend a proper system for bridge inspection. About one-tenth of the bridges upon our line are iron and the balance are of wood. We endeavor to have a skillful inspector, one skilled in bridge construction examine every structure at least twice each year, and this inspection is not construed as in any way relieving bridge carpenters, road masters and section foremen from making periodic examination and inspection of the same structures. Plans of every structure, including culverts, are on record in the engineer department. All bridges are designated by even numbers and culverts by the odd numbers. Each division superintendent is furnished a book giving the general dimensions and number of bridges and culverts on his division. The general superintendent and general manager have a book of the same kind. A book account is opened in the engineer department with each structure, in which is entered the date of inspection, repairs required, date when repairs are made, and the cost of same. The division foremen are required to state at the end of each month what has been done in making such repairs, so that in the end we have positive knowledge of when every stick

of timber or other material is placed in the structure. This you will observe requires a deal of clerical work besides that of inspection; and I was somewhat amused at receiving a request from a railway commissioner of one of the States through which our line passes, for a copy of all our reports of the inspection of bridges. I hardly need to say that we did not send them; we could not see that a carload of records for the use of a State railway commissioner would be of any service either to him or to us. Under this system of inspection it is gratifying to state that our line has been operated for twenty-two years without the failure of a single structure under normal conditions of traffic. If there is any class of persons that I do love, it is the average railway commissioner, appointed either by the governors of the different States or through political influence. If it is correct policy for States to appoint inspectors of railway structures and require the railway companies to conform to their demands, then it is but fair that the State be held responsible for any failure. I cannot believe that any State can be found willing to assume this responsibility and I do assume that no railway company will accept the inspection of a political officer, but will depend upon the inspection of those they believe to be their own tried and faithful servants. I believe in proper legislation that will require periodic inspection by competent persons appointed by the railway companies, and, in case of failure, hold, as the law does hold, the company responsible for any damage that occurs through fault of its own. Perhaps I have been more fortunate than many of my professional brethren. I have yet to find the manager of a line of railway who had been informed by the engineer that a structure was unsafe who did not immediately say, Make it safe at once.

Theodore Cooper, M. Am. Soc. C. E.—The question of the best method of securing a proper inspection of bridges is a very important one.

A bridge is a machine, and the man who inspects it must know what that machine is intended to do and whether it is in a condition to do it safely. It is not simply a question of the material of which it is made or of any individual member, but of each and every member individually and collectively.

Great stress is made in many cases of testing the iron or steel of which a structure is made, and but little attention paid to the design, proportion or assemblage of the parts.

Of course I do not mean to decry the testing of the material, but I do not recall at present a single case where a bridge has fallen solely because the material was poor.

The majority of failures have been due to bad design, bad workmanship or neglect to keep the structure in a proper condition to do its duty.

A bridge inspector must be an all-round man, sufficiently acquainted with the question of strains and theory of the action of each part of the structures under the varying conditions of the loads, with practical knowledge of the workmanship of the various kinds which enter into bridge construction, possessing an observing eye and a cultivated sense of feeling, and practical common sense, in order to determine whether a structure is doing its work satisfactorily and safely. He must take up each structure as a special problem and examine each member and each joint to see if it is in proper condition to do its full duty. He must not depend upon the office plans or upon the strain sheets. I have had to condemn bridges as dangerous after a field examination, that appeared perfectly satisfactory from the office plans. I might give many instances, but will mention only two. One was due to a change in the form of the castings, the webs taking the main part of the load being misplaced by the foundry man, so the castings were sheared through and the bridge on the point of collapse. In another case, only discovered by the merest accident, I found that the hanger of the end floor beam of a long span was carrying the same load as the end main diagonals, three panel loads, through the shop error of the pin hole being bored one-quarter inch larger than the pin.

We hear a good deal about defective rivets, and I have known structures where the rivets in certain parts were constantly being renewed. Many inspectors consider their duty done when bad rivets are cut out and new ones put in. But they should go further and determine why these rivets constantly work loose, and apply the remedy.

Soon after the Bussey bridge disaster I read in the papers that the railroad commission in one or more of the States had ordered all bridges tested for deflection to determine if they were safe.

Such inspection does more harm than good. Nothing in regard to the strength of a bridge can be determined by the deflection. The deflection of a bridge is simply the summation of the relative action of the several parts of the structure, good, bad and indifferent. Any one of dozens of the members may be at a point of absolute rupture without producing any appreciable change in the deflection.

How could the deflection of the Bussey bridge determine that the floor beam hangers were actually half broken and dangerous?

As to the subject of legislation, I do not think any law could be framed which would accomplish the purpose by defining the character or time of inspection. Neither do I believe any good would be gained by having the inspection made by a State expert. How would you define an expert? What is to-day the legal definition of an engineer? And even assume that the best bridge man of the day was made this expert, how long would it be before he would become an old fogy and oppose all progress beyond the stage of the art as he left it? No, we have not

yet learned all that is to be learned in any branch of science, and it is not according to American instincts to cease advancing and be bound to the dictum of any one man. Railroad bridges are not the only bridges where proper care and inspection are needed to protect life and property. Our cities, towns and counties have dangerous bridges. I know of bridges in one of our large cities that are so grossly neglected that if not now dangerous, they are rapidly becoming so.

There is, to my mind, only one way to apply a remedy. Let the law clearly and definitely make the officials of our railroads, cities and counties criminally responsible for loss or injury to life, commencing at the top and working down through the necessary organization until each stage of the organization can show that proper methods have been taken to know the condition of each structure at all times and that the power and means have been given to put and keep the same in a safe condition.

If a bridge goes down because the officials preferred not to know officially its dangerous condition, or if knowing, they neglected to remedy it, the criminal responsibility should be placed upon them.

If through a bad organization or false economy the overworked switchman, operator or despatcher produced a collision, the criminal responsibility should be on the officials to whom the bad organization or false ecomony is due and not on the man who has tried to do more than nature would permit.

Financial responsibility may be powerful enough in many cases, but it will not cover that large class of roads which have no owners, the footballs of Wall street—roads which are managed by men who have only a temporary interest in the same—roads whose officials have no higher aim than a good showing for the existing year or quarter, and who are willing to obtain such favorable temporary reports at the greatest hazards. Such roads will not spend money in a proper organization or for strengthening weak bridges. Criminal responsibility only can reach such officials.

Now, I am perfectly conscious of the vastness of a railroad organization, and that it is impossible for the directors or the operating officials to know, of personal knowledge, all the details of the different departments of their road. But it is their duty to know that each department is in charge of a competent specialist; to furnish him the men and means to perform his duties; and to know through proper system that his department is in good condition for a safe service.

Any attempt to establish a standard for the strength of bridges would be injudicious and impolitic. We will all agree that it is desirable that our bridges be built strong enough for any possible future traffic, and we also know that an increased capacity will not increase the cost anything like in proportion. But many well-managed, but financially poor roads cannot bear even this moderate increase in expenditure.

While it is their duty to the public to maintain their structures in a safe condition, it would not be right to compel them in their struggle for existence to do more than this. It is our duty as engineers to give them safe structures, and after that the best we can for their money. For roads capable of bearing the expense I would always recommend the best structures as the most economical in time, but for roads with short purses I would strive to give them perfectly safe structures and the best that their means would afford.

In regard to bridge floors, I cannot see any merit in a buckle-plate floor.

The floor of a bridge is the most important part of the structure and I favor a very heavy floor system. I believe in putting plenty of timber on that floor and substantial outside wooden guard rails as close to the rails as possible and as high as the rolling stock will permit.

If the sustaining members of the floor, stringers and cross floor beams are sufficiently strong, I do not see where the buckle plates will have any advantage over a good close timber floor, and if these sustaining members are not strong enough I do not see that the buckle plates will help the matter.

In addition to the wooden outside guard rails, which should never be omitted, an inside guard rail is also very desirable. Theoretically the inside guard rail is the better of the two, where both are of the same height. Unfortunately the inside guard rail must generally be limited to the height of the rails, which reduces its value very much, when acting alone.

The purpose of a guard rail being twofold, first to catch a derailed truck as soon as possible and prevent its slewing, and second to protect the trusses of a bridge from being struck by any part of the rolling stock, it is desirable that they be placed as near as possible to the rails. To prevent the wheels working up these guards, and surmounting them, it is desirable to have them as high as possible. I have seen on a guard rail, about 7 inches high, the marks of a derailed wheel, which was safely carried across the bridge, but which was continuously mounting nearly to its top by the friction.

The greatest obstacle to placing a guard rail 8 to 9 inches high, within less than 18 inches of the rail is the snow-plow.

There are plenty of bridges where the guard rails are placed so far out, in order to clear the snow-plows, that they serve no good purpose to either catch the truck before it slews to a dangerous extent, or to prevent the cars striking the trusses. Now, the snow-plow is a very necessary implement, but that the safety of so many bridges should be uninsured through neglect to make these snow-plows capable of passing automatically over the guard rails of bridges is a matter of astonishment. I would be surprised at any master mechanic who would state that snow-plows could not be made to automatically lift over such guard rails, at

a very moderate expenditure compared to the risks now incurred by rendering useless the guards over so many of our bridges.

I will finish by taking exception to a remark that has been made, comparing this Society, as regards establishing standards, to the Society of Master Car Builders; the case is not similar. The Society of Master Car Builders is composed of men who are devoted to the one subject of car building, and are therefore competent to consider and decide upon any matters pertaining to this business. Now, while we are all civil engineers in the broad sense of that title, our practice and experience are widely diversified, and we really belong to widely different professions. There are in this room electrical, hydraulic, mechanical, municipal, railroad and other special classes of engineers. Are the electrical or hydraulic engineers competent, as a class, to decide upon matters or standards for railroad purposes, or is the railroad engineer competent to determine standards for electrical purposes?

No, this society, as a body, is not competent to decide upon matters relating to any specialty.

I do not therefore consider it competent to establish standards upon bridge matters. Let us have full and free discussions upon all matters pertaining to the broad profession of the civil engineer, but leave the detail to those among us who are specially devoted to each subordinate branch.

H. Stanley Goodwin, M. Am. Soc. C. E.—Mr. Chairman, I think that almost everything I would say has been well said by those who have preceded me. I will say, however, that I agree with Mr. Wilson's plan and approve his suggestions. His plan of classification by numbers is certainly a most admirable one.

I think the schemes of legislation spoken of are impracticable, and some of them may, with propriety, be called Utopian. It has been stated that there are in the State of Michigan some 6 000 bridges; how is one engineer to look after the details of those 6 000 bridges? And if Congress is to take up this matter how is Congress to look after intelligently and practically, the construction and maintenance of 2 500 miles of bridges? It seems to me quite impracticable. Suppose it were to happen that all the bridges in the State of Michigan, or in the United States were to come under the control of one corporation, how would that corporation manage to care for these bridges? Why, simply in some such way as Mr. Wilson suggests. But one general head or superintendent could not give his attention to the investigation in detail, because there would be too many bridges. He would probably district the States under his supervision, and appoint to each district enough inspectors to insure thorough inspection and reports of all the bridges in that district. For the State to undertake to do all this, or to do such part of it as would be of use to the public, would seem to me going

beyond the authority which ought to be vested in State Government. I think that the success of the railroad corporations and of all branches of business in the country, is due to the fact that they are managed and governed by the interests in whose ownership they are, and not by State interference, and I think it would be desirable that the present state of affairs should continue. I do think, however, that there could be such a change in legislation as would prescribe what the railroad companies should do in the way of inspection of their bridges, and specifying that the companies should have such reports made to them that they would know their bridges to be sufficiently strong to carry the tonnage and loads that were passing over them. I do not think any railroad company would object to that.

There are a good many other points brought out by the circular. One is, and a good one, that this idea of inspection of bridges ought to include the turnpike and wagon road bridges. If Government interference is going to take hold at all, it should take hold of these. The public have to drive and walk over them, and they generally receive very little inspection, while railroad bridges are subject to an inspection more or less complete.

In regard to a standard bridge floor it does not seem to me practicable to adopt a standard form, because there is necessarily such a great variation in the styles of bridges.

In regard to overhead crossings, if it is meant to ask whether a grade crossing now existing and which has been in use for a number of years, ought to be altered to an overhead crossing, I would say that in many places overhead crossings are almost impracticable on account of their great cost, and would be very objectionable on account of the heavy grades of the overhead road.

There are many roads with small capital which would be very much embarrassed by a law making necessary such a change at all their crossings.

A. M. Wellington, M. Am. Soc. C. E.—Mr. Chairman, I have very little to say because I am not prepared to say it; the questions cover too broad a field to be all discussed together advantageously, but I must say I sympathize with what the gentlemen have said in regard to the question of legislation. I have remarked that in the written communications they have referred largely to the subject of legislation; I do not see what useful action we can take in regard to that. The only point as to which this Society should be particularly well qualified to express valuable opinions, is in specifying what are the ends to be accomplished, and that was very little touched upon in comparison to legal methods for providing a system by which bridges should be inspected. But in the other direction I think that there are strong evidences that something ought to be done, viz., in elaborating methods by which it

shall be ensured that iron bridges shall be built so strong in the first place as to be in no danger of breaking down. It seems to me that this is a far better method of insuring their safety; to build them so that they do not need to be gone over with a microscope once in six months, to prevent their giving out. Now, the fact is that nearly all our iron bridges prove worthless from having sailed too close to the wind in their design; the average life of an iron bridge is very brief; long before they are worn out they are removed because they are too weak. And moreover, I do not believe that we have reached the limit of load by any means. There are roads now that are running cars weighing 3 600 pounds to the foot, and the tendency of the introduction of train brakes will be to cause a further increase of load. In New York, for example, on the New York Central Railroad, the bridges are proportioned for light engines; they claim that they cannot use heavy engines, because with their easy grades the train becomes so long that it breaks in two, but with the introduction of automatic brakes, and with the introduction of train brakes that difficulty will be eliminated. It is only a question of a few years when those bridges will have to sustain very much heavier engine and train loads, and hence all prove worthless. I think our bridge specialists, who show such ability in design of details, do not realize the imminence of this change, and do not make sufficient efforts to provide for it in advance. I have no doubt that by the discussion in detail of these subjects which have been proposed, one by one, some very useful results may be accomplished, both in the way of economy and of improvement, but I hardly feel disposed to attempt to cover all that might be said in regard to them, or as to the numerous points in which I should differ more or less from the views so far advanced until the questions come up for discussion one by one.

Percival Roberts, Jr., M. Am. Soc. C. E.—There has been a great deal said in regard to the inspection of bridges in use, but there is an old adage: "You must first catch your hare before you cook it," and I think the inspection should commence from the day of asking for proposals on the bridge. I think, too often that when serious interests are involved, the sum total of the dollars and cents of the estimate are more closely looked at than are the interests themselves. Not only should estimates for bridges be compared one with another, but a thorough examination of strain sheet should be made by a thoroughly competent engineer. It is not sufficient to issue proposals based on certain standard specifications, but the strain sheets of all bidders should have careful attention. Now, presuming that this is not the case, however, I think that the first inspection we should have is the shop inspection. We all know how insufficient a shop inspection often is. We are very apt to find there a gentleman who has come to inspect a certain bridge, who walks into the shop with a tack-hammer in his hand, and

for several days after that you will hear the tack-hammer going all over the shop, and possibly there may be a check mark here and there on a rivet, etc. Sometimes, when the work is over, the inspector will ask for a copy of the drawings that he will examine later. I think that we should have a rigid and intelligent inspection while the structure is passing through the shop; that is the point where we can discover any defects. I think that all manufacturers will welcome such an inspection; it is a preventive to cutting prices, and to getting out a structure which is not up to the requirements of the specifications. The railroads too frequently start at the bottom with their inspectors, and instead of having an intelligent man for an inspector he is a young fellow, fresh from college, which I think is poor economy.

Mr. WHITTEMORE.—I fear what Mr. Roberts says is too true. In this matter of inspection I freely admit that I have at several times been most grievously swindled, and I judge others have also, as for instance: About a month ago I was at a shop where an inspector was supposed to be performing his duties, and the manufacturer said to me, "I wish you to look at that fellow, he is an inspector;" he was reading a dime novel in an inner office at the shops; presently some one came in and said, "that car is loaded now," and the inspector went out to the car, looked at the scales on which it was weighed, made a memorandum and then went back to his dime novel.

Mr. P. ROBERTS, Jr.—I understand that a number of foreign shops and works employ a man whose duty it is to entertain inspectors. They are professional experts in a number of things not pertaining to bridge work; they talk professionally to the inspector, take him out to dinner, etc., and it is a very good thing indeed.

The CHAIRMAN, Mr. T. C. CLARKE.—I believe that there are several inspectors present. I certainly think that you will all agree with me that they ought to be heard from.

C. C. SCHNEIDER, M. Am. Soc. C. E.—Mr. Chairman, I would say that the best inspectors are not always the ones who are the best appreciated; sometimes the men who have the least experience get the best positions and the good men have to take a back seat.

The CHAIRMAN.—I will ask any inspector to give us his views. They would be very interesting.

C. FRANK ALLEN, M. Am. Soc. C. E.—I am not an inspector, but I will say that I fear that this matter of legislation would relieve the railroad companies of responsibility. I believe that they should be held strictly responsible, both legally and to public opinion, but I believe that a certain amount of legislation would be valuable in the direction which we are aiming at. It seems to me that legislation should require some report upon the inspection of works so that the inspec-

tions that were made should become public property; so that it should be possible, when any defect was found, to run it down and fix the responsibility upon some one, either upon the president of the road, or upon the inspector. I believe that legislation that would secure some results of that sort would be healthy legislation, and could be thoroughly indorsed. One of the valuable points would be the fixing of the responsibility; I think we could make decided gains in that way.

There is one other view of the matter which is really, perhaps, of the most importance and yet most difficult to reach, and that is, that the general managers of railroads are frequently men who have a very small opinion of the value of the services of civil engineers; with a proportion of the managers the civil engineer is simply a surveyor; a very considerable proportion of railroad managers are prejudiced against civil engineers; if it is necessary to hire one they will hire the cheapest man that will do that service. This Society could do a good work in educating a considerable portion of our railroad men as to what a civil engineer is, and as to the value of his services; that when it is necessary to employ an engineer, they should get a good one, one who will know what the expenses to be incurred may be, the injury to car wheels, etc.; if it is necessary to run out a location, they should have a man of ability and experience, so that the company will be saved some money by listening to a man who knows what he is talking about. It is necessary to educate the railroad managers to the idea that the civil engineer will save them some money; if you touch the pocket you touch a tender point.

The SECRETARY.—A gentleman has handed me a copy of the provisions of the law of the State of Massachusetts with a circular from the Board of Railroad Commissioners of that State. It is interesting in this discussion as being a requirement of the State of Massachusetts through its legislative enactment of the present year.

COMMONWEALTH OF MASSACHUSETTS,
BOARD OF RAILROAD COMMISSIONERS,
20 BEACON STREET.

BOSTON, ....., 188...

To..... .....Railroad Company.

SIR:

Your attention is called to the following sections of Chapter 334 of the Acts of the present year, being "An Act relating to the Examination of Railroad Bridges:"

SEC. 1.—Every railroad corporation shall, when requested by the railroad commissioners and at least once in two years, have an examination of its bridges and the approaches thereto made by a competent and experienced engineer, who shall report to the corporation the results of his examinations, his conclusions and recommendations, and the corporation shall forthwith transmit a copy of the report to the board of railroad commissioners. The first report shall be made and transmitted to the board not later than the first day of November, in the year eighteen hundred and eighty-seven, and subsequent reports shall be made and

transmitted at intervals of not more than two years. When a railroad corporation builds a new bridge it shall forthwith have a report in like manner made and transmitted to the board. The reports shall furnish such information in such detail and with such drawings or prints as may be in writing requested by the board of railroad commissioners.

SEC. 3.—Nothing herein contained shall be construed to exempt a corporation from making other and more frequent examinations of its bridges and the approaches thereto.

In accordance therewith, the Board of Railroad Commissioners request you to transmit to them, on or before the first day of November next, the following information, plans, etc., relating to the bridge structures on the line of your railroad and its branches, said information, etc., to be given for every bridge structure of over ten feet opening in the clear, between abutments, but not to include highway or street bridges over the road.

I. A statement, in tabular form, giving in order the following data: (1) number of bridge; (2) town in which, and (3) line upon which it is situated; (4) precise location upon the line, if on record; (5) direction and distance from, and name of nearest station; (6) ordinary name; (7) nature of crossing (stream, street, etc.); (8) number of openings, and clear span of each; (9) length over all; (10) material; (11) general style of bridge; (12) whether deck or through; (13) approximate maximum height of rail above stream, street, etc.; (14) date of erection; (15) names of designer and of builder; (16) by whom erected; (17) whether or not built to definite specifications. On this sheet shall also be stated the weights of the heaviest engines, tenders, and loaded cars at present in use on the road, specifying load on each axle, and distances apart.

This table is preferably not to be a blue print, so that additions can be made to it from time to time, and is to be on one or more sheets of uniform size. Sample sheet for the above returns can be seen at the office of the Board.

II. A "strain sheet" for each structure, giving—

(a) For frame structures, the calculated maximum stress in each piece;

(b) For plate-girders and beams (including floor beams and stringers of truss bridges), the maximum moment and shear at points not over ten feet apart, including the center and ends.

The loads to be assumed in making out the strain sheet are to be either those under IV (a), or the heaviest loads in actual use on the road—whichever cause the greatest stresses—and are to be clearly shown or stated on the strain sheet. The actual loads in use may, if desired, be considered in the calculation as replaced by a suitable uniform load headed by a suitable concentrated load.

If practicable, the information called for under IV (b), (c) and (d) shall all be given on the strain sheet.

III. Blue prints or drawings showing complete dimensions of each structure, with sections and dimensions of every part, and details of all connections or splices. Scales for drawings to be: (a) for general elevations, cross-sections and plan, not less than $\frac{1}{4}$ inch to the foot; (b) for all details of connections and splices, and sections of parts, not less than $\frac{1}{2}$ inch to the foot, and preferably not less than $\frac{3}{4}$ inch to the foot. The complete floor system is to be clearly shown on these drawings.

These sheets, together with the strain sheet and the table under I., shall be made of uniform size, 28″ × 40″, including a margin on the left hand side of 2 inches.

IV. For each bridge: (a) If built to definite specifications, a copy of such specifications, together with a statement as to the tests, superintendence, etc., by which they were enforced.

(b) If built without definite specifications, a statement of the source and quality of the material employed, whether any tests were made upon it, together with the results of such tests.

(c) A statement of the frequency with which it has been inspected, and by whom; and, so far as known, the results of any tests applied to it since its erection.

(d) If not given under (a), a statement of the loads for which it was calculated, and of the stresses allowed per square inch under different circumstances.

V. A report by a competent and experienced engineer, as required by Sec. 1 of the Act referred to above, which report shall include the results of his examinations, his conclusions and recommendations, not only in regard to the bridge structure itself, but also in regard to the approaches to the bridge, and the piers and abutments.

Plans not conforming to the above requirements, if already prepared, may be presented to the Board for approval.

George G. Crocker,
Edward W. Kinsley,
Everett A. Stevens,
Commissioners.

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Frederic Graff, Past President Am. Soc. C. E.—Mr. Chairman, I wish to speak of a matter which seems to me to be connected with this subject. The City of Philadelphia has been agitated frequently by the question of elevated railroads, and I would like to ask from the gentlemen here what inspections are insisted upon for the systems of elevated railroads in the City of New York?

R. I. Sloan, M. Am. Soc. C. E.—The elevated railway structures in the City of New York were designed by careful engineers, and built and erected by responsible bridge building firms. Good iron was used in the construction, and good engineering work was done in the beginning. A large portion of the system has been in operation more than nine years, and the entire system of 32 miles over eight years, and over 12 000 000 trains have passed over the roads. The structures have been submitted to continuous use under the light rolling loads. The strains imposed upon the girders do not exceed in any case what good iron ought to stand.

The factor of safety required by the Rapid Transit Commissioners and the daily thorough inspection maintained render the structures perfectly safe.

The maintenance of the structure, track, signals and buildings is in charge of the chief engineer, his assistants, and the road master and his assistants, four supervisors, and the structure and track foreman.

Daily reports are made by the road master to the chief engineer, who reports the same day to the general manager, all defects in foundations, structure or track. We have also arranged with Mr. Albert Lucius, M. Am. Soc. C. E., an expert bridge engineer, to make frequent examinations of the structures and report to the chief engineer every two or three months.

The great increase of travel on the Third and Sixth avenue lines has made it necessary to increase the weights of the engines from 17 to 19 and 21½ tons, and to keep the factor of safety within the limit prescribed by the Rapid Transit Commissioners, we have strengthened the floor beams on the Sixth avenue, pin-connected structure and strengthened the Third avenue girders by a double system of triangulation, the braces united at their intersections. The 5 x 10 inside guard timbers have been replaced by timbers 6 x 8 inches, also an additional timber has been put alongside the outer rail, making the guard 12 x 8 inches. Fifty-pound rails are being replaced by 70-pound rails. The pounding of the rail joints has been lessened by the use of improved supports and splices. The Fisher joint has so far given very good satisfaction by presenting to the tread of the wheel, when passing from one rail to another, an even surface.

The greatest metal wear is on the steel rails of the sharp curves, those of 90-feet radius. These have to be renewed about every 18 months. The ties have shown very little decay, and have been down from eight to ten years. Many are shortened and broken up by the spikes, bolts, log screws, etc. All those taken out are replaced by ties six inches deep and eight inches wide.

Mr. Graff.—You are in the condition of the bridge structures, you have not reached the standard for the proper elevated road, but you are searching for it by strengthening every part?

Mr. Sloan.—I would recommend the use of plate girders for a proper elevated road. We all know that the plate girder if properly proportioned is economical, stiff, trustworthy and requires less inspection than any other form of short span girders.

A Member.—What is the size of the plate girders?

Mr. Sloan.—Those plate girders lately put in are 39 inches deep. Top chords 6 x 6 angles, 65 pounds to the yard, web ⅜ inches. Those on the old Ninth avenue structure and in Battery Park are about 24 inches deep, but are very stiff, seldom find a loose nut.

J. Foster Flagg, M. Am. Soc. C. E.—The ties are not creosoted?

Mr. Sloan.—No, sir; three years ago we tried some which had been vulcanized.

Mr. Flagg.—Have you tried broken-jointed rails?

Mr. Sloan.—Yes, sir; we break the rail joints.

Mr. Flagg.—Do you use the bevel joint?

Mr. Sloan.—We tried the bevel joint for a short distance on the Sixth

avenue, but I would not attempt that again. I would prefer to have a square rail joint.

A MEMBER.—Do you find any defects apparent in the columns?

Mr. SLOAN.—None in the rectangular columns. We found some two years ago, during very cold weather, a few rivets had burst in some of the high Phœnix columns which were filled with cement mortar. I think it was caused by the contraction of the iron segments which form the column hugging close the rigid cement column inside. Each segment as it contracted pulled away from the others so hard as to burst the head from the rivet. I did not consider there was danger to the structure, as the surplus strength of the columns is very great.

J. J. R. CROES, M. Am. Soc. C. E.—What is the weight of the engines?

Mr. SLOAN.—From 15 to 21½ tons, according to the type.

Mr. CROES.—During the busy hours what is the interval between the trains on the Third avenue road?

Mr. SLOAN.—About two minutes, but that is at 129th street.

Speaking of grade crossings, the interlocking apparatus we use at Chatham square has demonstrated what can be done in passing trains if carefully managed. About 3 000 000 trains have passed over the crossings at this point, and there has been but one collision; that occurred last fall. A train coming from Second avenue collided with a northbound train going up Third avenue; the engine driver ran past the red or danger signal.

The CHAIRMAN.—Since you have been strengthening the Third avenue line have you seen any loosening of rivets?

Mr. SLOAN.—I have heard of none on the girders which have been reinforced. Where the structure is painted a light color a loose rivet is soon discovered by the little streak of rust which shows on the light paint.

Mr. CROES.—How long is it expected to continue the strengthening?

Mr. SLOAN.—We commenced about three years and a-half ago, and will have the Third avenue line done by next spring.

Mr. CROES.—How many trains run over the Third avenue road a day?

Mr. SLOAN.—On an average 490 going north and 490 coming south.

Mr. CROES.—I remember a few months ago there was a set of switches at South Ferry, and the man in charge said there were 2 400 trains a day passed over them.

I think if iron bridges are good for anything at all that the two lines of iron bridges in New York ought to show their defects and their merits. I do not suppose that any iron structure in the world is subjected to the tests that these are.

THEODORE COOPER, M. Am. Soc. C. E.—In a paper by Mr. Benjamin Baker, of London, an eminent engineer, upon the strength of materials

subject to alternate and repeated strains, he makes the statement that the elevated roads of New York City have developed unexpected defects due to the frequent application of the loading. There is no truth in this statement. I was connected with these roads during their early stages, and have ever since made their action a special subject of study and observation. I had nothing to do with their design or proportions, but did have an intimate knowledge of their construction.

In some cases, charter time agreements necessitated the temporary acceptance of imperfect work. This was one cause of the renewal of certain portions. Another was the unexpected development of the traffic. Locomotives capable of handling two cars were supposed to be ample, whereas now trains of five cars are inadequate to satisfy the demands. To provide for this, much increased strength has had to be provided. We can also say from our present knowledge of the duties to be performed by these roads, that the design is not the best.

There have, however, been no defects developed in these structures which cannot be clearly traced to faulty design, hasty workmanship or want of appreciation of the traffic demands. And not a bit of evidence can be found to show that the rapidity or number of trains have developed defects that would not in time have been produced under other conditions of the same loads.

I do not believe that any number of applications of the loading will produce any more injury than a single loading upon a structure so proportioned that the actual strains shall never exceed the elastic capacity of the material. There is no reason why an elevated railroad cannot be designed and constructed to be as permanent as any other structure made by man, and as safe as any other system of railroad.

Mr. William Kent.—I am not a bridge expert, nor a member of this Society, but, through the courtesy extended to the American Society of Mechanical Engineers, am attending your convention.

In regard to the competency of this Society to pass upon the question as to what legal measures should be taken, one gentleman says this Society is not competent because it contains several professions. There is no Society to-day especially devoted to bridges. This Society certainly contains within itself all the best bridge talent of the country, and if they are only five, I say this question of bridges had better be referred to them for discussion. The work done recently by the American Society of Mechanical Engineers on the question of standard steam boilers and trials of standard pipe threads was this: the Society appointed two committees, one of boiler experts and one on pipe threads. The boiler report was submitted and discussed, and when it came up finally before the Society they recommended that it should be printed by the Society; after a very lengthy discussion, two years ago, it was decided that the Society should indorse no report and should take no action

whatever, but that they would publish this report with the names of the committee attached. It was the general opinion that the report would have more weight than a mere vote of the whole Society. When the question of pipe threads came up it was again decided that the Society would not adopt the report, but would publish it. Now all the steam-users are adopting it and it is going to be the standard of this country.

Applying this argument to the bridge question, I say that this whole subject of what measures, legal or other, can be taken had better be referred by the Society to a committee of its own members, who should work for one year, and that their report should then be published.

Mr. D. J. Whittemore.—In the main I agree with the conclusions of Mr. Bouscaren as given previously in this discussion. I cannot, however, bring myself to the belief that different roads and their branches may have bridges of different ultimate strengths. Railways are operated by superintendents, not by engineers. While a superintendent may be well posted as to the capacity of structures upon the various branches of a line of road of which he has charge, he may be displaced at any time and a new superintendent appointed who may assume at once that all structures within his jurisdiction are of the same strength, and he is liable to send a train of maximum weight over any portion of his line. Of this I can cite an instance: An individual owning a patent car designed for carrying an enormous load had it loaded to its fullest capacity at one end of our line, and the division superintendent, without inquiry, started it on the way to its destination. It so happened when its journey was about half reached that the matter came to my knowledge, and while we would have been able to have safely transported the car to its destination, upon my representation of the matter to the general superintendent, its progress was arrested and one-half its load removed. General superintendents understand these matters pretty thoroughly, but they are not always able to control the action of division superintendents who are younger men, therefore I am strongly of the opinion that every company should have its structures on all its lines strong enough to safely sustain the heaviest traffic of the day.

I think it quite right that our States should enact laws requiring railway companies to have periodical inspection of structures by competent persons and also require said companies to use devices that are proved to conduce to the public safety, and we know the laws inflict proper penalties in case of their non-use, but I do object most strenuously to having men, calling themselves engineers, appointed, through political influence and receiving their appointments through State officials or by election from the people and not selected by the railway company itself, which is or should be a competent judge of capacity, to tell us how to do our business. An engineer appointed by a railway company has, or should have, his heart in the business, feel responsible for the safety

of every structure, and should he have a conviction that any structure on his division was faulty, his conscience would not permit him to neglect the matter until it was righted. Generally, I think that the railway companies of any magnitude in our country have, or are now inaugurating, a pretty thorough system of inspection of their structures; therefore, I say, let the legislation on this subject be general, and of such a character as will impel the railway companies to have thorough and competent inspection of their structures by their agents only. States should not enter upon this matter without being willing to assume responsibility, through neglect of their own agents.

While we have had many and valuable suggestions as to the inspection of bridges, we are anxious to ascertain the opinion of members as to what should be considered a proper rolling load in designing bridges for the present and future traffic. I am sure that superintendents and managers of railways will continue to increase the size and weight of carriages and the load carried thereon, until they find that tie and rail become insufficient and not an element of economy, and in doing this they will go beyond the limit of economy, as railways are now constructed. A general expression upon this subject will guide many engineers in their designs hereafter.

In regard to standard floor, I think that a universal floor cannot be well adopted for this reason: conditions are different in different parts of the country. Southern roads on our continent do not have to contend with the snow that we meet with in the northern States and Territories. In the northwestern States, where my field of work lies, and where the temperature is no more severe than is found in the New England States, I have seen snow so deep that men when shoveling out the track rested themselves by sitting down on the top of telegraph poles; and I presume there are several here present who have had analogous experiences in the northwest. While the climate in its general features is not widely different from that of our eastern States, the wide, open, rolling prairies permit what are termed blizzards, that pile snow to enormous heights. Without due consideration of these matters I commenced some years ago building the floor mentioned by Mr. Pegram, and put guard rails 9 to 10 inches outside of the rail; in some instances we lined this with iron. Since that time we found it necessary to construct between fifty and sixty snow-plows, or about one to every hundred miles, and any number of what we call flangers, and to operate these snow-plows and flangers safely, it has been necessary to place the guard rails instead of 10 inches outside of the rail to from 14 to 17 inches. While it may be well to recommend the Latimer guard on many of the lines of this country, I fear it will be difficult to introduce it on all. In regard to grade crossings, I am sure that it is not policy for this Society to express an opinion; density of population will control this matter.

When our company built into Minneapolis it was a small place, and we went across streets at nearly grade; now since the city has grown to its present size we are about to lower many of the crossings below the railway or above, as may seem proper. Had we been compelled to do this at the outset it would have been many years before that place would have secured a railway to it. The volume of traffic and importance of a highway for the public using the same, determines what is policy in such cases, and ordinarily the railway companies and the public are able to settle this matter satisfactorily.

Robert Moore, M. Am. Soc. C. E.—I sympathize with Mr. Whittemore in what he says about the excessive loads which are put upon bridges. I think that the branch lines might perhaps co-operate with the engineer if he adopted a lower standard in his bridges, yet he can have no assurance that some abnormal load will not be put upon them. I am therefore very much in favor of building bridges, even of branch lines, up to a high standard, provided the money can be obtained; for, of course, this is, after all, the controlling consideration. But if you cannot do any better, I would build bridges to carry 2 000 pounds per foot, which is the old standard to which bridges used to be built.

Speaking of abnormal loads, I would mention the case of the transportation of our street cables. Within the last two or three weeks we have had pass over the bridge at St. Louis a number of these cables, of which the last one weighed over 90 000 pounds, and was on a car which could not have weighed less than 30 000 more, giving for a 30-foot car a load of 4 000 pounds per lineal foot, which is much too great for most bridges. It is, in fact, a load which bridges ought not to have to stand, as the damage to the track of even our best roads caused by loads like these is so great as to make them anything but economical.

Mr. D. J. Whittemore.—It seems to me that there is another view to take of this matter—Mr. McAlpine can correct me if I am wrong, and that is, that in the early inception of railways in this country they could be operated after the rails were laid, and then the civil engineer was no longer required. Generally, as soon as an engine could pass over the road the civil engineer was dispensed with as soon as possible. Oftentimes he was looked upon during the construction as a necessary evil, one who spent money but did not earn any. In this way many railways were operated for about seven years, and by that time ran into bankruptcy. The successors of the bankrupt company then were forced to the conclusion that they required a civil engineer to direct, and replace the decayed and worn-out structures, and often employed the cheapest man they could get, brave enough to confer upon himself the title of civil engineer, to act as such.

W. J. McAlpine, Past President Am. Soc. C. E.—I want to say that I commenced engineering at a salary of $10 a month and board.

Mr. Whittemore.—Nearly every railway company of magnitude on our continent at the present time, I am happy to say, finds it necessary to employ the best talent it can secure not only in the engineer service, but in every other department of railway service. Now there can be found men of as much culture, honesty and energy as can be found in any avocation in life; and any railway company that has not these factors cannot serve its stockholders properly. It is entirely unnecessary for me to make such remarks before the members of this Society, but we must remember that our proceedings are sometimes perused by men, not engineers, interested in railways.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVII.—December, 1887.)

# INSPECTORS AND BRIDGE WORK.

By SAMUEL TOBIAS WAGNER, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JULY 2D, 1887.

## WITH DISCUSSION.

The object of this paper is to discuss the characteristics and work of inspectors, especially as regards their bearing on the bridge industries of this country, and to determine upon the most harmonious relations between them and the manufacturer.

Inspectors may be divided into two broad but eminently distinct classes, viz.:

*First.*—Those who know what they want, and know when they have it.

*Second.*—Those who do not know exactly what they want, and therefore are often at a loss to decide when they have it.

Manufacturers may also be divided into two classes, viz.:

I. Those who intend to give good work, who have a reputation at stake, and who value it.

II. Those who will resort to underhand dealings in order to pass off bad work, and whose object is to make money at the expense of their reputation.

The combinations of these four classes, under various conditions, is the principal subject of this paper; and although no definite rules can be laid down as to methods of procedure, nevertheless it is possible to adjust certain points which will tend towards a clearer understanding as to the scope and jurisdiction of both manufacturer and inspector.

In bridge work the percentage of inspectors per pound of iron is increasing very rapidly, and this fact makes it all the more important that a thorough understanding of the scope of inspectors be reached, as upon them depends, to a very large extent, the tranquil or ruffled close of a contract.

All railroads of any magnitude have their regular corps of inspectors, or special inspectors, under the chief or a consulting engineer, and bridges of any consequence which are left without an inspector are fast becoming things of the past.

The object of an inspector is to determine whether the contractor is living up to his specifications. The specifications should always be as complete as possible, and no contract should be let without them. There is nothing so unsatisfactory for both contractor and inspector as to try to accomplish an uncertainty in a satisfactory manner, and if they succeed in getting along well the conditions must be most favorable.

No contract should be signed without specifications, if anything is to be required and demanded, and neither party in signing should lose sight of what they are agreeing to do. I dare say many contracts are signed with perhaps merely a glance at the specifications, and then when the contractor, in the hurry of his work, is reminded by the inspector that some certain clause is not being complied with (of the existence of which, perhaps, he has never heard), it is sure to make trouble, especially if it is expensive to remedy. This at once makes unpleasant relations between the two, and only one party is to blame.

An inspector should not only have specifications and know them by heart, but should understand exactly what is meant, and be capable of saying whether the work, as performed, is in accordance with them or not. If his inspection is not to be final (a condition of things which should never happen), he should know what he has a right to accept and what to reject. There should be but one inspection of raw material, one inspection of work leaving the shops, and one of the finished structure, and each should be final for what it was made.

An excellent plan to be observed throughout a contract is that of having all material that has been accepted from the mills stamped by the inspector. In the same way the inspector's stamp should appear upon every finished bridge member, and thus show that it has been accepted.

It is very unfortunate, both for railroad companies and manufacturers, that there are inexperienced inspectors for bridge work. It too often happens that men of no theoretical or practical training in any branch of engineering, are deemed competent to decide upon the merits of work when they do not know the principles of its construction, and it is this class of inspectors, under certain circumstances, who are very annoying to the contractor and cause him most trouble.

A man's disposition determines, in a large measure, his success as an inspector. A naturally suspicious man makes his own life a burden, besides being a source of needless worry to all around him; while one free from suspicion and with even an ordinary amount of common sense and intelligence can get good work and make no trouble even if his technical knowledge is small.

If an inspector is determined to make things unpleasant he can always succeed in so doing, as his power for the time is necessarily great; but unless his reasons are good he commands little consideration from the contractor, and soon becomes unpopular.

The contractor, upon beginning work, should notify the inspector at once of the fact, in order that he may avoid delays and unnecessary expense.

In the standard specifications for material and workmanship of iron and steel structures adopted in this country by the leading bridge builders, a clause is introduced bearing upon this point which is excellent; part is herewith quoted:

13. "The inspection and tests of material will be made promptly upon its being rolled, and the quality determined before it leaves the rolling mill. All necessary facilities for this purpose shall be afforded by the manufacturer, but if the inspector is not present to make the necessary tests, after due notice is given him, then the contractor shall proceed to make such number of tests as may have been agreed upon," etc.

By this means all raw material reaching the bridge shops will have been inspected and tested, and no extra handling should be allowed as is now too often the case. In the inspection of the raw material from

the rolling mill the inspector should pass upon the straightness of the individual pieces, if he will not allow means of remedying this defect afterwards.

In many of the English and French shipyards not a pound of iron or steel will be accepted without the inspector's stamp, and if this rule were adhered to more closely in America, much trouble and serious expensive delays could be avoided.

Assuming that all the iron for a bridge has been rolled, tested and inspected for surface defects, straightness, etc., and has been delivered at the shops, the question then arises as to what inspection should be made as the work progresses, and where the same should be examined. As this question is difficult of solution, it may be well to briefly divide up the different classes of work on a bridge, and to take an ordinary through span as an example.

We may divide the work upon this into:

| | | |
|---|---|---|
| 1. Riveted members. | Floor beams. | Intermediate posts. |
| | Stringers. | Struts. |
| | Chords and end posts. | Pedestals and wall-plates. |
| 2. Forgings..... | Eye-bars. | Upset and eye rods. |
| | Clevises, and other blacksmith work. | |
| 3. Machine work..... | Pins, rollers, turned bolts, etc. | |

We can omit any discussion of sections 2 and 3, as any inspection before the final one is entirely unnecessary, except, perhaps, an examination into the methods of manufacture if special methods are required in the specifications. In case no special operations are demanded, the inspector has no right to insist on a special method being adopted by the contractor, if, in the end, the work is satisfactory.

Under the head of riveted members so many requirements are included that we cannot arrive at so rapid a solution. Most specifications do and should cover almost all the separate steps in manufacture from the raw material to the finished member.

Assuming, for example, a section of the top chord of a bridge, let us follow it through the shop. It is punched, riveted, faced and bored, and about each operation something is usually specified.

In the punching the only necessary point for the inspector to satisfy himself upon is the relation between the punch die and rivet to be used. If this is not specified no inspection is necessary, but it is apt to appear in almost all cases.

In the assembling of the punched parts for riveting, he may have to satisfy himself that the work has been properly punched, and that no gouging of holes is allowed. Unless his contract is small, or he has plenty of assistance he will not be able to devote much of his time to this part of the work. At this point the inexperienced inspector usually brings up the drift question, and some arguments are usually necessary to convince him that it is as yet harmless.

When the member is being riveted he may with propriety watch the drift, and as soon as the member is completely riveted should pass (by marking) upon the quality of the riveting and straightness of the member. The member is now faced and bored and the final inspection made for length and distance between pin holes, and the final stamp placed upon it, after which it is painted or oiled and ready to ship.

It seems to be a disputed point among inspectors whether these final measurements are necessary, as in some cases they omit them entirely, while in others the manufacturer is checked on every point. It seems to the writer that these extremely careful measurements are unnecessary, especially if the same contractor erects the structure. Of course the inspector may desire to check them up for his own satisfaction, and it no doubt eases his conscience.

The instrument generally used by those who use and check the final measurements is the steel tape, and each inspector making the measurements wants his tape taken as the standard. For the purpose of checking measurements within one-eighth of an inch the steel tape will answer fairly well, but further than this it should not be relied upon. For finer checks than one-eighth of an inch the shop standards should be used, as it must be assumed that the whole bridge will be built from them.

For these fine measurements the writer prefers standard steel poles.

If the inspection is not made on riveted members as above, the manufacturer may be seriously delayed if all the inspections mentioned are intended to be carried out and a very strict inspector on hand. There should therefore be a distinct understanding between contractor and inspector as to just what points are to be specially inspected and the work arranged in the shops to suit. If the inspection is to be rigid upon straightness, then the work should be inspected as soon as it has been riveted.

On the other hand, if the contractor understands just what quality of

workmanship is required, then the danger in having one final inspection is lessened, but not entirely averted, whereas with some inspectors it is not safe to proceed more than a step at a time.

To insure prompt execution of the work the inspector should pass upon it promptly and decidedly, and should know just what to give and take in case of a question on a minor point. An inspector who adopts a standard within the reach of mortal man, and sticks to it, has less trouble with the manufacturer than one who is changing constantly. The two arrive at a distinct understanding and each know when they have passed the line.

To insure pleasant relations between a rigid inspector with detailed specifications, and a contractor with a reputation, the inspector on the one hand must:

*First.*—Have had some experience, and have and exercise common sense, besides thoroughly understanding his specifications.

*Second.*—Be on hand promptly and see all the material tested and inspected.

*Third.*—Have a distinct understanding with the contractor as to what will be expected.

*Fourth.*—Be on hand while the work is progressing on his contract and keep himself informed as to when he will be wanted.

*Fifth.*—Inspect the work as it progresses, and not leave it until it is ready to ship.

*Sixth.*—In case of a piece of questionable work, reject it for the moment if other work is awaiting inspection, and decide finally at a more opportune time. The writer has seen an inspector halt and deliberate for a whole day over one piece of work, while other urgent work was awaiting his inspection.

On the other hand, the contractor, in order to carry out his part of the programme, must:

I.—Determine to live up to his specifications as he understands them.

II.—Arrange with the inspector in detail where the different inspections of the material and work are to be made.

III.—Give the inspector ample notice when he will be wanted.

Perhaps it would be advisable to have printed forms as given below to be filled out by the contractor and inspector at the beginning of a contract, and each retain a copy.

INSPECTION OF RAW MATERIAL.

....., 18..

*Contractor's Order.* *Inspector's Order.*

| DESCRIPTION. | TO BE MADE AT. | TESTS REQUIRED. |
|---|---|---|
| Universal plates ..... | | |
| Sheared plates..... | | |
| Angles ..... | | |
| Channels, etc..... | | |
| Bar iron..... | | |
| Pin iron..... | | |
| Castings..... | | |
| Other shapes ..... | | |

Contractor.....
Inspector.....

This would have the effect of systematizing the testing and the location at which the material is to be made, thus furnishing in a proper manner a great deal of information, which the inspector has ordinarily to find out from time to time as best he can.

It will be impossible to follow any regular course for the riveted work, as everything depends upon circumstances; but the writer cannot too strongly urge that the inspection should in no case be left until the work is entirely finished, except where absolutely necessary, especially if inspector and contractor are strangers. It is admitted that this method of examining the work at various stages in its progress makes more work for the inspector, but if everything is to be examined it is merely a question of when it is to be done. Besides, if any alterations are to be made they cannot be made too soon in the progress of the work, as they are easier remedied and the damage done to the piece made very much less.

A very unfortunate case is that of rejection of work by an inspector which the contractor feels is unjust, because the only appeal is to the inspector's superior, and in nearly every case he is sure to be sustained in his decision. If the inspector determines not to yield, the contractor in most cases must give up the fight. This is one of the most exasperating positions for him to be placed in, as he virtually has no redress. Could not a clause be inserted in specifications providing for arbitration in a case like this?

As it now stands, the only thing a contractor can do is to be very careful in signing a contract to ascertain who the inspector of the work is to be, and if the same is well known for giving trouble, throw up the contract unless another is substituted. This method is in existence; its use is unpleasant, and it is very much to be regretted that there is apparently no better way out of the difficulty.

A rather remarkable condition of things may here be noted: As good work is almost invariably obtained by inspectors who have no serious difficulty with contractors, as by those who are constantly at variance with them, and it will be found in most of these cases that the reason is their uniformly just decisions, thus giving the contractor no ground for complaint.

An inspector's decision once given should be final, and if he does not use a stamp it is advisable that he should accept everything in writing, although the stamp is much the preferable way.

The writer, although unwillingly, is in favor of a black list of troublesome inspectors to be circulated among bridge-builders, but feels that if the matter were to receive proper attention the most serious points that lie as stumbling blocks between inspector and contractor could be removed by a system of arbitration; that this would place the blame where it was due, and thus avoid troubles that, if not thickening, are growing none the less.

## DISCUSSION.

Mr. J. B. Johnson, M. Am. Soc. C. E.—I would like to ask whether Mr. Wagner can get better results with steel measuring rods than with steel tapes. In my experience the steel tape is the most accurate way of measuring that has yet been devised.

S. T. Wagner, M. Am. Soc. C. E.—In answer to that, it has been our experience that steel tapes expand or contract quicker than the mass of the metal they are on. And it is one disadvantage that most of the steel tapes are divided rather crudely; it is very seldom that you see a steel tape divided into less than an inch. In the case of the steel rod, at least one such as I have seen used, the division is very sharp, and when they are only divided into feet, the inch and fractions of an inch can be measured off very closely with the standard 12-inch steel rule. I have seen recently some steel tapes which are divided into sixteenths of an inch, and in which the divisions are very sharp. As for these I cannot say how they would work, but a great many inspectors have brought steel tapes and have called attention to the fact of certain

differences, but the marks were not exact and when we have measured them we found that we could not agree with the inspectors. In the works where I am engaged they have discarded the use of the steel tapes except for measurements not including fractions of an inch.

Mr. J. B. Johnson.—This brings up the general subject of long standards of length. There are many difficulties with long standards of any kind. The universal difficulty of temperature is an additional difficulty with a long standard, affecting its absolute length. There are no two standards that will agree and the trouble increases with the length. Now, it seems to me there should be a greater effort to obtain accuracy in long standards, say in 50-foot or 100-foot tapes or chains, but in tapes rather than in chains. At present we can only rely on the maker's brand, and heretofore there have been few facilities in this country for getting absolute lengths of tapes. It is now possible to do that. There are now in St. Louis two tapes, 300 feet long each, which are absolute standards. They are absolute lengths that have been determined by comparison with United States base lines so that the lengths are known with an accuracy of one-millionth part of the length. Whether it be the standard meter bar of the French archives, or whether it be the base line 4 or 5 miles in length, there never has been any means devised for getting the absolute length over about the one-millionth part; but that may be done with the steel tape, and the steel tape may be used for getting other lengths with that same degree of accuracy. It is now absolutely proven by the measurement of base lines on the Missouri River Survey, the details of which are printed in the Report of the Missouri River Commission for 1886 and 1887, that base lines can be measured with a steel tape as accurately as has ever been done with the most elaborate apparatus ever invented. So that men who are engaged in the most accurate measurements are coming round to the steel tape as being the best and most accurate standard that has been used; therefore, I think that it should not be dismissed from such work as this too hastily. The probability is that if good results have not been obtained from it, it is because it has not been properly used. Of course it should have accuracy of subdivision.

It seems to me that if a small tape, say $\frac{1}{8}$ of an inch wide and $\frac{1}{64}$ of an inch thick, of about 0.002 of a square inch in cross section could be laid upon an iron member, it would very soon reach the temperature of that member; the small tape laid upon the iron member would arrive at the exact temperature of that member; that is a result that is to be desired. If you take the large iron rod it would not arrive at the temperature of the member in a considerable time. Of course we must know the various constants of the tape, but when the tape is compared with some other standard tape you then get these facts. If intelligently used, the steel tape is destined to become, I believe, the most valuable means of accurate measurement available to engineers.

A. M. WELLINGTON, M. Am. Soc. C. E.—Mr. Chairman, did I understand, that with these 300-foot steel tapes, base lines can be measured as accurately as by the cast-iron apparatus?

MR. JOHNSON.—Yes, sir.

MR. WELLINGTON.—In any weather?

MR. JOHNSON.—Not in ordinary weather, but in selected weather. You must give the tape a fair show, and you must get the actual temperature of that tape, say to a half, or a tenth, of a degree, but that whole ques ion has been thoroughly investigated. If the base line be measured four times and scratches on zinc strips made for each 300 feet, and the temperature is not the same on any two measurements, then these marks do not correspond.

The comparative values are only known after they have been reduced. You have then an absolute check for each 300 feet, and that furnishes a good opportunity for obtaining the probable error. With proper precautions, in the best weather, a probable error of one in one million is readily obtained. The tapes are supported on a series of wire hooks, every 20, or 30, or 40 feet; if these hooks are not placed at the same distance apart, then, of course, the effect of the hanging loops must be introduced, but that submits readily to accurate computation. The tape is stretched by a weight which hangs by a lever so that there is no friction in getting the stretch. The thermometer is of the same temperature as the tape because the whole atmosphere is of the same temperature. I did not mean to call up the matter of base-line measurements, but it serves as an illustration to show that the engineer should not dismiss the use of the tape too summarily.

J. F. FLAGG, M. Am. Soc. C. E.—I would like to ask if, after the length of the tape has been ascertained by comparing it with the base line, whether that was afterwards used for measuring another base line to see how it compared with that? After you have obtained the length of the tape, whether it is tried on another base line.

MR. J. B. JOHNSON.—That has been done in Sweden, but not yet in this country. It is to be done soon; another base-line is soon to be measured that is of the same standard of accuracy. * The tape should be used on a still and cloudy or foggy day, because it is a very sensitive standard, and its temperature could not otherwise be accurately determined by a mercurial thermometer.

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* In 1886 one-half of the Olney Base Line, in Southern Illinois, was measured three times with the Mississippi River Commission Tape. The length measured about 10 816 feet. In 1887 the entire base line was measured twice with the same tape, the length being about 21 633 feet. From these five measurements there resulted the following values for the length of the tape.

| FEET. | | |
|---|---|---|
| 1. 299.07469<br>2. 299.07581<br>3. 299.07532 | Mean of first measurement. | =299.07527 |
| 4. 299.07528<br>5. 299.07522 | Mean of second measurement. | =299.07525 |

See Report Missouri River Commission for 1887.

There are now in this country two absolute standards, each 300 feet long. One is the property of the Mississippi River Commission, and the other is the property of Washington University, St. Louis. The co-efficient of expansion of the Commission's tape was determined by myself by continuous observations for four days and three nights, with average temperature of some fifty degrees. Micrometer microscopes were mounted on fixed pillars over the ends of the tape, which was stretched by a 16-pound weight. The entire tape was inclosed in a wooden box, and suspended from hooks every 16 feet. The stretching weight was slung from a noose, which was kept adjusted to an angle of 45 degrees with the vertical. Its co-efficient was found to be 0.00000699$\pm$4 in the last place. Its modulus of elasticity was found to be 27 400 000. Its absolute length was obtained by measuring the Olney base line.

The Washington University tape was standardized by comparisons at two widely different temperatures with that of the Mississippi River Commission. I may say that co-efficients of expansion of steel standards of length have been found to vary from 0 0000048 to 0.0000070, a variation of nearly 50 per cent.

F. Collingwood, M. Am. Soc. C. E.—How do you preserve the accuracy of the length of the tape?

Mr. J. B. Johnson.—I do not understand that the mere fact of winding the tape changes its length. After you exceed its elastic limit its length is increased, but a steel tape is so thin that it never takes a permanent set by winding upon a reel.

Mr. Collingwood.—Of large size?

Mr. Johnson.—The reel is about 6 inches in diameter.

Mr. Collingwood.—Is that a small diameter?

Mr. Johnson.—Medium, I should say. The tape is $\frac{1}{8}$ of an inch wide, and about $\frac{1}{64}$ of an inch thick.

Mr. Wagner.—I would like to say, that from what I know of steel tapes it is not so much the steel tape itself as the number of steel tapes that has to be contended with. The inspector comes and complains that the work is long or short by his tape. It is mainly with this that we experience the trouble. If steel tapes are properly used and properly devised, then, with proper care, very good work can be obtained, but each inspector comes along and says that all the work has got to be done by his tape.

J. J. R. Croes, M. Am. Soc. C. E.—Would it not be better for the manufacturer to furnish the steel tape to the inspectors? The cost would not be very great and the manufacturer would thus give the inspector the standard by which all the work would be laid out. In that way harmony could be preserved.

Eliot C. Clarke, M. Am. Soc. C. E.—I would like to ask, whether the use of wood for accurate measurement is safe? I know it has a bad reputation; but in measuring lines under the Detroit River, we used rods

10 feet long. We selected with great care some pieces of black walnut, and, after having them very thoroughly boiled in oil, made four rods, putting little metal strips at each end, and we had great success with them; we never had any trouble from moisture of the wood. It seems to me that wood, properly prepared, for many purposes would answer very well.

Mr. J. B. Johnson.—If the measurements are not to be made with the utmost accuracy, wood, I think, is a very satisfactory standard, but for extreme accuracy it will not do. I have compared 3-meter painted pine level rods, with absolute standards, for weeks at a time, by means of micrometer microscopes, so that the facilities were good for getting very accurate results. The rods were thoroughly tested and were treated exactly alike, and were stood vertically upon one end, so that there should be no good reason why they should change their lengths; but each rod had a different curve of changes; they were not reliable.

A Member.—How large were they?

Mr. Johnson.—About 10 feet long. The extreme changes amounted to about half a millimeter; that is, about 1 in 6 000. The graduations were on the surface, however, and the changes noted might have resulted from deflections. End comparisons, or comparisons on the neutral axis, might have shown no change.

Mr. Collingwood.—Did the gentleman intend to say that the inspector's opinion should, in any case, be final?

Mr. Wagner.—Yes, sir; I do think that when an inspector passes upon a piece of work he should not have the right to reject it afterwards, unless there is something of serious consequence; he ought not to have the right to change his opinion.

H. B. Seaman, M. Am. Soc. C. E.—If the fault is not seen by the first man it will be condemned by the second.

Mr. Wagner.—If the work is bad it ought to be condemned immediately. If there is anything wrong that would injure it it ought to be condemned by anybody interested in it; but if there is only a minor point in view, I think the contractor should not be put to any expense after the inspector has once passed the work.

Mr. Collingwood.—I think that very often the contractor would like to appeal from the inspector's decision, and I think that a contractor ought to have that right. But in some cases there may have been something which could not be seen at the time which can be seen later on.

Mr. Wagner.—I agree with the gentleman fully.

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